[Inset, to precede p. 1, Nov. 1940 Journal.

JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

No. 1. 1940–41.
NOVEMBER 1940.

ORDINARY MEETING,

5 November, 1940.

SIR CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A., the retiring President, in the Chair.

Sir Clement Hindley said that it was, he knew, with great regret that the members of The Institution had heard of the deaths, during the recess, of Sir Robert Hadfield and Sir Joseph Thomson. Resolutions of condolence had been passed by the Council, and had been sent to their respective families.

Sir Clement announced that the Council, being of opinion that it was not in the interest of The Institution that the present King of Italy and the present King of the Belgians should remain Honorary Members, acting under the provisions of the Royal Charter, had ordered the removal of those names from the Institution Roll.

The Council reported that they had recently transferred to the class of

Members.

John Selby Balfour, B.A. (*Cantab.*).
David Webster Jones, B.Sc. (*Leeds*).
Harry Peretz Kaufman, B.Sc. (Eng.) (*Lond.*).
Harold Edmund Manning, B.Sc. (Eng.) (*Lond.*).
David Crull Milne, B.Sc. (*Glas.*).
William Morris.
Joseph Parkin.
Cyril Harry Sands, B.Sc. (Eng.) (*Lond.*).
Sydney Upton.

and had admitted as

Students.

Patrick John Anderson.
George Herbert Armitage.
John Ledger Arnold.
Roy Pennington Ashton.
Clifford Aveyard.
Alan Fleetwood Badman.
Merrik Burrell Baggallay.
John Howard Bailey.
Frank William Baldrey.
Gilbert Nigel Ballard.
Trevor Howard Barker.
William Nowell Barlow.
Richard Ryall Barnes.
Eric Barton.
Robert Ratcliffe Beaumont.
Roy Best, B.Sc. (Eng.) (*Lond.*).
George Henry Bisacre, B.A. (*Cantab.*).
Laurence Bridgewater.
Joseph Andrew Broda.
Leslie Broughton.
Ian Ganson Budge.
William John Henry Burstow.
Eugene Gabriel Clarke, B.Sc. (*Belfast*).
Donald Arthur Cox.
George Deryk Cribbes.
Walter Scott Land Dalgleish.

Richard Angel Delacroix.
William Richards Dexter.
Francis George Dobbs.
Edgar Wilfred East.
Roy Ernest William Elderton, B.Sc. (Eng.) (*Lond.*).
Anthony Henry Hetherington Emmerson.
William Duncan Fisher, B.Sc. (*Leeds*).
William Herbert Forrester.
Raymond Franklin.
Bernard Frederick Fox, B.Sc. (*Leeds*).
Eric Garner.
Peter George Garnett.
Joseph Norman Gass.
John Stuart Glenny.
Roy Green, B.Sc. (Eng.) (*Lond.*).
Leslie Maurice Hall.
John Lorimer Hammond.
Neil Gilbert Hansen.
David Alan Harris.
Albert Richard Harris.
Ross Lindley Harrison.
Harry Trevor Hind.
Ralph Vernon Hines.
Jack Leslie Holland.
Richard Maxwell Hollis.
Peter Charles Gerald Isaac.
John William Jackson.
John Paul Johnson.
Frank Andrew Scott Johnston.
Arthur Alfred Jones.
Thomas Llewelyn Jones.
Harold Kerr, Jun., B.Sc. (*Leeds*).
Thomas Donald Kershaw.
Gordon Geoffrey Knapp.
James Leggat, Jun.
George William Lindo.
Kenneth Forsythe Logie.
James Eric Lugg.
John Jamieson Maconochie.
Anthony Luckin McCormick.
Alexander James McLaren.
John Gordon McLellan.
James McMichael.
Raymond John Mare.
Aubyn Raphael Margary.
Alexander Ronald Miller.
Philip John Millman.
Basil Robert Monckton.
Sydney Muller, B.Sc. (*Leeds*).
John Lloyd Munro.
Donald Edgar Murchison.
Kashinath Vinayak Natu.
Jack Newcombe.
Peter John Newman.
Joseph Brown Newton.
Patrick Ivor Parker, B.Sc. (Eng.) (*Lond.*).
Norman Peel.
James John Trevor Pratt.
Dennis Charles Prideaux.
Francis George Godfrey Quayle.
Colin Race.
Stuart Edgar Radbourne.
Keith Ruthven Rands-Allen.
Dennis Victor Reeve.
George Eric Rice.
David Humphrey Roberts.
Malcolm Owen Rochfort.
Victor Theophile Schaerer, B.Sc. (*Witwatersrand*).
Robert Ross Shepherd.
Brian Malcolm Macdonald Sinclair.
William Henderson Skinner.
John Berridge Staunton.
George Bingham Stewart.
Donald Edward Street.
Bernard Leslie Swift.
Raymond Bromley Tabor.
Harold Taylor, B.Sc. (Eng.) (*Lond.*).
Dennis Mountford Taylor.
John Henry Tempest.
David Hamlyn Thomas.
Benjamin Franklyn Thomas.
Anthony Waude Thompson.
Charles George Trimmer.
Eric Courtney Turier.
Moinuddin Atiq Aziz Ahmed Vahidy.
Robert Anthony Wall.
Francis Walley, B.Sc. (*Bristol*).
Kenneth Harry Ross Wilson.
William Edward Winstanley.

The retiring President said that it was his very pleasant duty, before vacating the Presidential Chair, to have the honour of introducing to the members Sir Leopold Halliday Savile, K.C.B., the new President.

Sir Leopold, he said, was a man with very great experience and a practical engineer of high reputation when at the Admiralty and later in partnership with Sir Alexander Gibb. Apart from that, he had served on the Council for 10 or 11 years, and had given a great deal of his time and energy and interest to the affairs of The Institution. Sir Clement was quite sure that the Presidency would be perfectly safe in Sir Leopold Savile's hands and he asked the members to give him all their loyal support.

Sir LEOPOLD HALLIDAY SAVILE, K.C.B., President, having taken the Chair,

Mr. S. B. DONKIN, Past-President, proposed a vote of thanks to the retiring President. He observed that the duties which Sir Clement Hindley had performed during his term of office had been exceedingly arduous, although they had not been obvious to the members of The Institution. There had been very few meetings, and therefore the members did not perhaps realize how much work he had put in. The Council realized that, and appreciated what he had done to carry on the work of The Institution. In his opinion, and he was sure it was the opinion of the Council, Sir Clement Hindley was the best Committee Chairman that they had ever had ; he was a great organizer and administrator, and an executive head, and it was for those reasons that his work had been so extremely good. He had been for the past 12 years or more a Member of Council, and not more than 7 or 8 years ago he, with others, had started reforms in The Institution to make it more useful to its members. Sir Clement was then and had been ever since, an arduous worker for those reforms, particularly in regard to publications, By-Laws, relations with the public, and lastly in connexion with research. All those things had been going on since those reforms were instituted. He had helped during his year of office to keep The Institution in the forefront of all engineering activities, and he was now leading the Council to organize and perhaps to report on those vital problems of reconstruction and of an improved social order, all of which were essential, when the war finished, to aid democracy. He formally proposed a vote of thanks to Sir Clement Hindley for the services he had rendered to The Institution during the past year.

Colonel J. R. DAVIDSON seconded the motion, which was carried by acclamation.

Sir CLEMENT HINDLEY, in acknowledging the resolution, expressed his appreciation of the vote of thanks and said that he came almost with an apology for having done so little because in those times it had been very hard to see a way for action, so that he really finished his term of office with the feeling, "so little done, so much to do." That feeling was uppermost in his mind and he really wished that there had been more opportunities of serving The Institution during the past year. Nevertheless, they could feel reasonably satisfied that The Institution had progressed during those difficult times and had kept an even keel and achieved something. In conclusion he urged the members to give all their loyalty to the President, to see to it that whatever was done, was done in the interests of the technical and ethical standards to which they held : those were the things which would continue through any kind of trouble.

The President then delivered his Presidential Address.

Mr. F. E. Wentworth-Shields proposed a vote of thanks to Sir Leopold Savile for his Address and said he was well known for the work he had done in supervising Admiralty harbours all over the world and commercial harbours everywhere. He moved that the best thanks of The Institution be accorded to the President for his Address and that he be asked permission for it to be printed in the Journal.

Dr. W. L. Lowe-Brown seconded the resolution, which was put to the meeting and carried by acclamation.

The President expressed his appreciation of the vote of thanks. He took the opportunity to express his gratitude to Mr. C. S. Barton for the researches he had made both in the ancient and modern literature, which had been of great assistance to him in preparing his Address. He thanked the meeting for the manner in which the Address had been received and had much pleasure in allowing it to be printed.

The meeting then closed.

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PRESIDENTIAL ADDRESS OF

SIR LEOPOLD HALLIDAY SAVILE, K.C.B.

PRESIDENT, 1940–41.

In ordinary years the Member upon whom you confer the privilege of election to the Presidency of this great Institution has some idea of the duties that lie before him and can console himself that where others have found themselves able to fulfil these duties with satisfaction to all concerned, he, with the guidance of Past-Presidents and the Council, and with the help of the Secretary and his staff, always so readily and so ably given, may hope to carry the great honour that you have entrusted to him in a manner that will not lower the high standard set and followed by his predecessors: but this is not an ordinary year, and nobody knows what part we may have to play in the struggle which we are waging against the forces of evil, as our share of the effort to bring about ultimate victory. All I can say is that we—and I know that I speak for the members as a body—will give without stint all of our energies, wherever required, to achieve that end. We have faced dangers in the past; we do not flinch from them now.

It is, however, perhaps a good thing during times of great stress occasionally to relax and to turn our thoughts right away from the present struggle. Therefore, when considering a suitable subject for my Address, I decided to follow the example set by Mr. W. J. E. Binnie in his Presidential Address 2 years ago and, leaving modern times, to touch upon ancient history. Since harbour engineering has been the branch of our profession with which I have been principally associated for most of my career, I propose to deal with harbours, from the dawn of written history to the early days of the Roman Empire.

The Four Harbours of Alexandria.

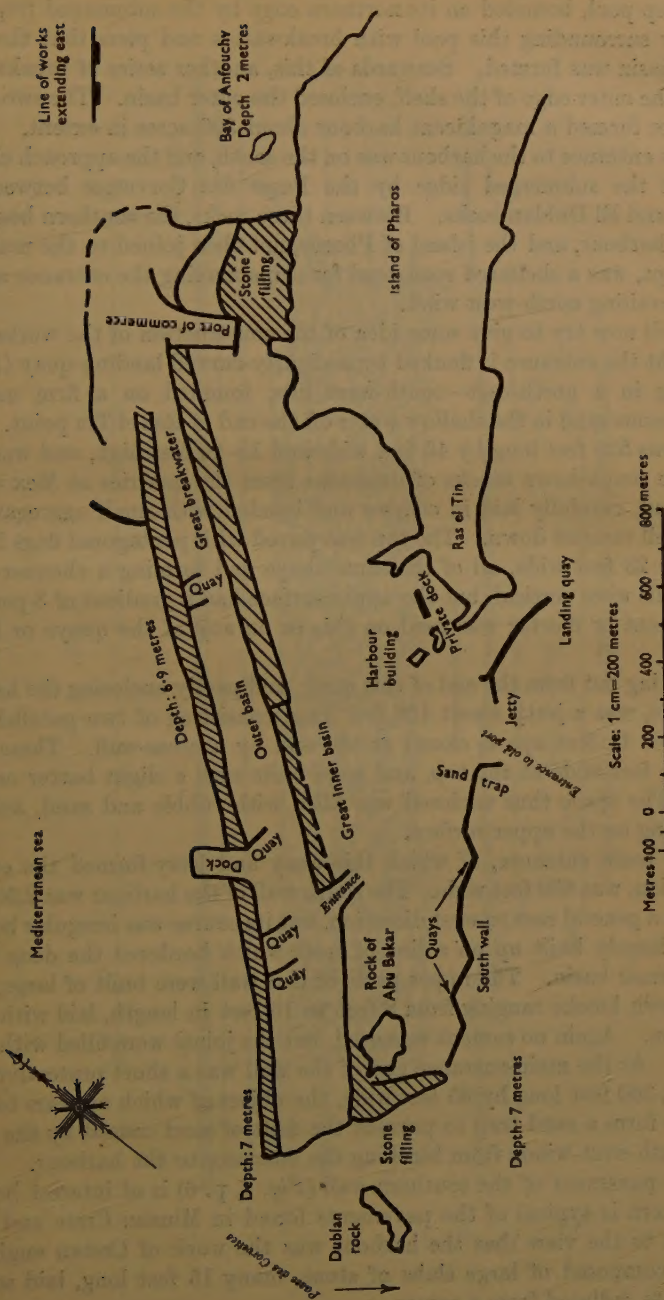
Shipbuilding and harbour engineering are two of the oldest branches of our profession. It is well established that before 3300 B.C. the

Egyptians built sea-going ships and that they made voyages to far lands to procure iron, lead, silver, and other materials ; and it is recorded on the Palermo stone that about 3000 B.C. King Seneferu built sixty great ships to go to the Syrian coast to bring cedar-wood for his works. In the British Museum is a stone statue of Bedja, son of Ankhu, one of the great ship-builders of his days. The terminus of these voyages was on the Canopic branch of the Nile, where was situated A-ur or the Great Door, which Mr. P. E. Newberry calls " an ancient Alexandria of a period earlier than 3000 B.C." Little is known about this harbour, except that Narmer, one of the earliest kings of the First Dynasty, considered it of great importance and decided to conquer the petty kingdom of Harpoon, to which it belonged. It was an inland port and probably had the disadvantages of that type, especially as it lay on the banks of an arm of the delta. The actual site of the port is not known, but I refer to it because it is the earliest harbour of which I have found mention and because it marks the beginning of the harbour of Alexandria, which, I think, has the longest history of any harbour in the world. I propose to devote some of my time to a study of the great schemes adopted on the Alexandrian site over a period of nearly 5,000 years (Fig. 1, Plate 1, facing p. 10). There have been four distinct harbour building periods—the harbour of A-ur, about 3000 B.C. ; the great harbour of Pharos, soon after 2000 B.C.; the harbour of Alexander the Great, begun in 332 B.C. ; and the modern harbour, which dates from A.D. 1870.

The Great Harbour of Pharos (*Fig. 2*) was typical of the pre-hellenic form of massive structure, far more massive than some of the great harbours of modern times, and it is well worth study. Its layout and the skilful use made of the configuration of the bed of the sea might have been the work of a modern harbour engineer. " When," says M. Gaston Jondet, " one examines the largeness of the project and ponders on the boldness of its execution, it becomes obvious that it was conceived by a sovereign power of unequalled breadth of view, a realistic genius capable of conquering and keeping the mastery of the Mediterranean sea." Who the realistic genius was we do not know, for Egyptian history, curiously enough, has no record of this harbour. M. Raymond Weill attributes both its conception and its construction to the Minoan Cretans, who at that time were the greatest sea-faring power in the Mediterranean. It could not, however, have been made without the co-operation of the reigning Pharaoh, possibly Senusret of the Twelfth Dynasty, a famous builder of colossal buildings typical of the Egyptian, Minoan, and Mycenæan civilizations of those early times. This gives us a date somewhere between 2000 and 1800 B.C.

The harbour was based at its eastern end upon the island of Pharos, and at its western on the rock of Abu Bakar. It also took advantage of the submerged ridge running from Marabout point to the north of Pharos, and of the shelf which sloped from this towards the deep sea. From the bay of Ras el Tin at the western end of Pharos to the Abu Bakar rock there

Fig. 2.



PRE-HELLENIC PORT OF PHAROS, 1800 B.C.

is a deep pool, bounded on its northern edge by the submerged ridge. It was by surrounding this pool with breakwaters and piers that the great inner basin was formed. Seawards of this, another series of breakwaters, using the outer edge of the shelf, enclosed the outer basin. The two basins together formed a magnificent harbour about 300 acres in extent.

The entrance to the harbour was on the south, and the approach channel crossed the submerged ridge by the Passe des Corvettes between the Ikvan and El Dublan rocks. Between these rocks, the southern boundary of the harbour, and the island of Pharos, not then joined to the mainland of Egypt, was a sheltered roadstead for ships making the entrance against the prevailing north-west wind.

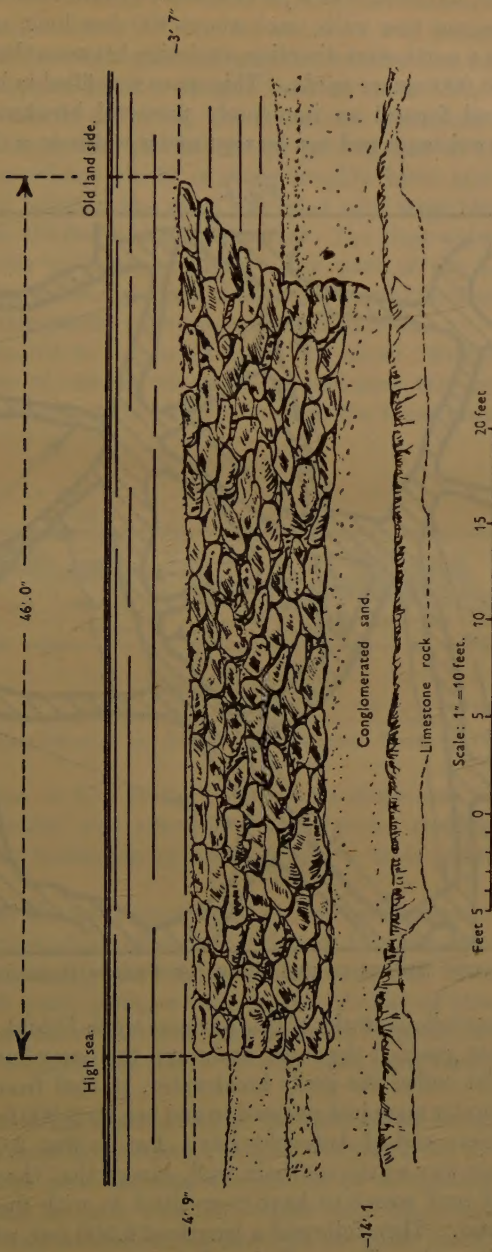
I will now try to give some idea of the construction of the works. On the right the entrance is flanked by a slightly-curved landing-quay (*Fig. 3*) running in a north-east—south-west line, founded on a firm mass of argillaceous sand in the shallow water off the end of Ras el Tin point. This quay was 525 feet long by 46 feet wide and 18–20 feet high, and was built of large rough-hewn blocks of limestone from the quarries at Mex on the mainland, carefully laid in courses and bonded with small aggregate and sand well tamped down. The top was paved with pentagonal flags 26 feet long by 23 feet wide, all of the same shape and forming a chequer-work. The walls were vertical, but the upper surface had a gradient of 3 per cent. No cement or mortar was used on this or on any of the quays or breakwaters.

Jutting out from the end of this quay, and partly enclosing the harbour entrance, was a jetty about 426 feet long, consisting of two parallel walls just over 41 feet apart, closed at the end by a cross-wall. These walls were 7½ feet wide at the top, and were built with a slight batter on each face. The space thus enclosed was filled with rubble and sand, and had no paving on the upper surface.

The main entrance, of which this quay and jetty formed the eastern protection, was 650 feet wide. The south wall of the harbour was 2,300 feet long, in a general east to west direction, but its course was irregular because it was largely built up on a line of reefs which bordered the deep water of the inner basin. The upper parts of this wall were built of large, carefully-hewn blocks ranging from 8 feet to 16 feet in length, laid with great precision. Again no cement was used, but the joints were filled with small stones. At the main-entrance end of the wall was a short protective mole or spur, 360 feet long by 65 feet wide, the object of which appears to have been to form a sand-trap to prevent the drift of sand caused by the south and south-west winds from blocking the entrance to the harbour.

The pavement of the southern wall (*Fig. 4*, p. 6) is of interest because its pattern is typical of the pavements found in Minoan Crete and lends support to the view that the harbour was the work of Cretan engineers. It was composed of large slabs of stone, many 16 feet long, laid so that the joints radiated from a centre.

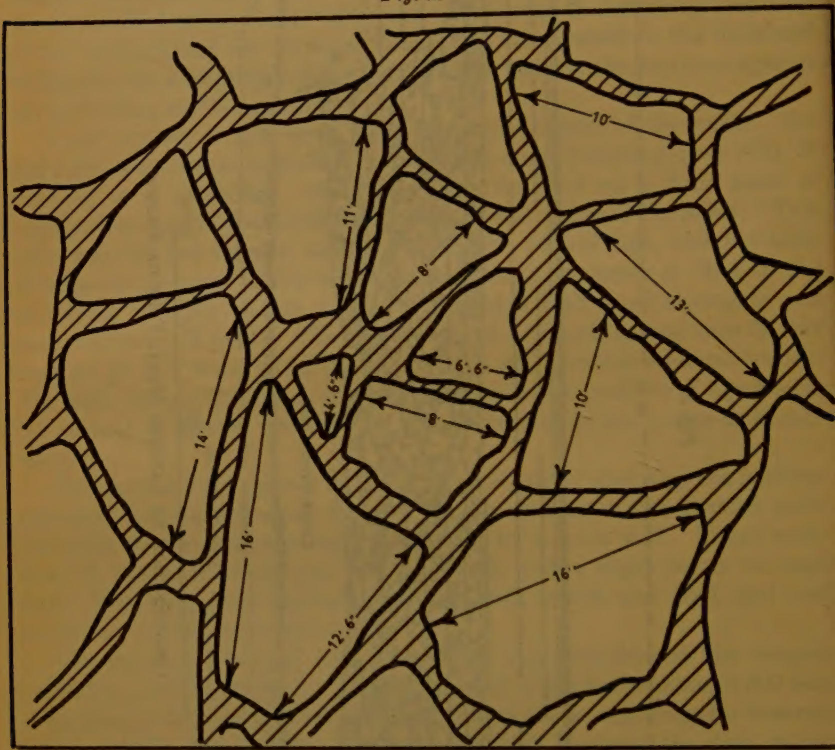
Fig. 3.



Cross-Section of Landing Quay, Port of Pharos.

The southern wall ended at a point a short distance south-west of Abu Bakar. Thence ran two walls, each about 490 feet long, one in a north and the other in a north-west direction, enclosing between them a triangular area of about 28,000 square yards. This space was filled in by large blocks of limestone and formed an immensely powerful breakwater, much of which is still in existence and can be seen under water on a clear day.

Fig. 4.



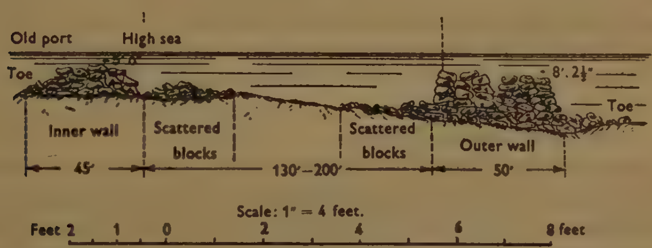
PAVING STONES OF THE QUAYS OF PHAROS HARBOUR.

The most marvellous works of this harbour were, I think, the two great breakwaters that guarded the inner basin and the outer basin. The first, which M. Jondet called the great breakwater, started from the northern end of the triangular mass just referred to and ran for 8,500 feet in a straight line to the western end of Anfouchy bay. For its first 2,000 feet it was built in the same way as the southern wall, except that the part bordering the Abu Bakar rock seems to have been filled in with dumped stone to form a solid mass. Then followed a length of 6,500 feet which needed to be very strongly made. Two walls founded on firm argillaceous sand overlying the submerged ridge already mentioned, were built 130–200 feet

apart (*Fig. 5*). Each ranged in width at its upper surface from 26 feet to 40 feet, and had a batter of 1 in 30, and each was protected by a substantial toe. Their height, judged from the remains that have been found under water, appears to have ranged from 20 to 30 feet. The depth of water in the basin is unknown, but it may be estimated at 25–40 feet, with considerably deeper patches in the pool of Ras el Tin. The walls were built of enormous blocks of stone roughly hewn and coarsely laid. All of the space between the walls was filled with large blocks, forming a surface between 180 feet and 250 feet wide. The great width would enable defending parties to move rapidly to any part of the harbour during piratical attacks, whilst in normal times it was useful for drying and repairing sails and fishing-nets, weaving ropes, and so forth.

Running parallel to this breakwater, and about 650 feet distant from it, was another of similar construction enclosing the outer basin, the entrance to which was by a passage through the inner breakwater a little

Fig. 5.



TYPICAL SECTION OF GREAT BREAKWATER, PORT OF PHAROS.

to the north-east of Abu Bakar, between its single-wall and double-wall portion. Protection was afforded by two moles running in the same direction as the landing-quay and protective mole guarding the main entrance.

The whole of the inner breakwater formed an immense quay. Besides this, several jetties about 60 metres (197 feet) long ran out from the outer breakwater, and nearly the whole of the south wall of the inner basin formed a broad quay, giving a total length of quay of about 10,000 feet. There was also a kind of dock built out seawards from the outer breakwater, the purpose of which is not clear. It may have been another entrance to the harbour.

The remains at the eastern end of the harbour bordering on Anfouchy bay are not so easy to interpret. About 650 feet from its end the great breakwater of the inner harbour was pierced by an opening 160 feet wide and 525 feet long, to form what M. Jondet calls the commercial harbour. This small port had two entrances, one from the outer basin, and one direct from the sea, carefully protected by two incurving breakwaters. Beyond the commercial harbour the great breakwater continued for a short distance

to the shallow water at the commencement of Anfouchy bay, where a north and south cross-wall closed the harbour. A very large area between the breakwater, the wall, and the shore of the island was filled in with large stone blocks, as at the west of Abu Bakar.

At the extremity of the point of Ras el Tin, near the main entrance to the inner basin, is a small island around which are the remains of other works, including a short mole which enclosed a small private dock—perhaps for the use of craft owned by the harbour authorities. This surmise is made the more probable by the fact that slightly to the north-west are the submerged ruins of a large building, more than 92 feet long by 46 wide, with approach channels and steps, which appears to have been the head-quarters of the port management, where pilots and the captains of ships would come to receive their orders.

To the east of the great harbour was a smaller one occupying the bay of Anfouchy. It also was protected by breakwaters and equipped with quays, but it afforded only a shallow depth of water and was used chiefly as a fishing-centre.

I have attempted to give a brief description of the ancient harbour of Pharos, as revealed by the researches of M. Gaston Jondet, carried out between 1910 and 1915; and when the science shown in its layout and construction is considered, we must, I think, agree with him that it was, indeed, the work of a realistic genius.

It may seem strange that when Alexander the Great founded Alexandria and built his harbour in 332 B.C. he should have taken no notice of these wonderful works. The reason was that they had disappeared under the sea, and all that marked the site of the future city was a little village at Rhacotis and a small colony of fishermen. There is no more record of its fall than of its rise. Homer may refer to it in the fourth book of the "Odyssey," where he describes Pharos as an island in the troubled sea having within it a haven with fair moorings. If this is so, then its decline must be dated some time after 1000 B.C.

A few words as to the cause of its disappearance may be interesting, although "disappearance" is really a misnomer, because, as M. Jondet has shown, a very large portion of the works still exists and on a calm day parts of them can be seen clearly below the surface of the sea. The ridge of high ground upon which the harbour was built is formed of limestone similar to that exposed in the quarries of Mex. Overlying the slopes of this ridge is a thin layer of clay, upon which is a thick layer of river silt in various states of consolidation. Covering this on the higher slopes is the stratum of hard argillaceous sand, and it was upon this that the walls and breakwaters were built. M. Jondet considers that, as the silt consolidated, its bearing value weakened and the stratum of sand which rested upon it glided down the slopes in sudden subsidences, the underlying clay acting as a sliding surface. The process was purely mechanical, although earth tremors may at times have started the movement. In this

manner whole portions of the works glided below water-level, often without any damage to their structure.

Fifteen hundred years after the foundation of the harbour of Pharos, Alexander the Great, returning down the Canopic branch of the Nile from his visit to the temple of Zeus Ammon in the oasis of Siwah, halted at the village of Rhacotis. Ever since his destruction of Tyre he had determined to build a harbour that should be her rival. At Rhacotis he had found the place he wanted. He, himself, is said to have traced the plan of Alexandria and its harbour, which his famous engineer, Dinocrates, was ordered to carry out (*Fig. 6*, p. 10). The main feature of the harbour was the great mole, 600 feet wide and 7 *stadia* (about 1 mile) in length, and hence called the *Heptastadion*, from the mainland to the island of Pharos, which divided the roadstead into two basins. It was built in a depth of water of 36 feet, and its construction entailed the excavation, transport, and deposition of about 2 million cubic yards of stone. The basin on the right of the mole formed the Great Harbour, and that on the left the *Eunostos* or Haven of Happy Return. Two openings through the mole connected them, thus conforming to the ancient rule that a harbour should have two entrances. The Great Harbour was bounded by the Lochias headland, the *Heptastadion*, and the eastern end of the island of Pharos. Seaward it was protected by a pier built out from Lochias and by a line of dangerous reefs, which made entrance to the harbour difficult. It was chiefly to remedy this that Ptolemy built the world-famous Pharos, or lighthouse, one of the seven wonders of the world, on the eastern point of the island. Alexander erred in putting his harbour in this place, since the depth of water was not so good as in the neighbouring haven, the reefs and Lochias pier did not provide sufficient protection against the winds, and the entrance was always difficult. Within the Great Harbour lies the small island of Antirrhodus, and between it, the mainland, and Lochias was formed a small *Portus Regius*, or Port Royal, for the king's ships. Between the *Portus Regius* and the *Heptastadion* the shore was lined with quays and storehouses. The public granaries were on the *Eunostos*, where also was a small inner harbour enclosed by piers. It was on this basin that the important canal connecting the harbours with lake Mareotis and the Nile, by its Canopic branch, opened. Alexandria partially fulfilled its founder's purpose of crippling the trade of Tyre; but this was due to the policy of Ptolemy Philadelphus (285–247 B.C.) who made a harbour at Berenice on the Red Sea, connected it with Coptos, on the Nile, by a road provided with water-places at proper stages, and reopened the canal between the Nile and the Red Sea at Suez. Thus he captured for Alexandria the important trade of the Indian Ocean and the Red Sea, which had hitherto passed by Eloth and Eziongebir to the coasts of Palestine, whence it was carried in Tyrian ships over the whole of the then known world. Alexandria's gain was Tyre's loss.

A period of more than 2,000 years passes (Fig. 1, Plate 1). In the mean-

time the sand, which the engineers of ancient Pharos had been so careful to fend from the entrance to their harbour, had passed along the roadstead

Fig. 6.

PLAN OF ALEXANDRIA AND ALEXANDER'S HARBOUR, 332 B.C.

and had been caught up by the *Heptastadion*. Gradually it broadened until it formed that belt between the waters upon which a large portion of the modern city of Alexandria is built. The engineers of 1870 dis-

PRESIDENTIAL ADDRESS.

FIG: 1.

ALEXANDRIA, SHOWING THE ANCIENT HARBOUR OF PHAROS, ALEXANDER'S HARBOUR, AND THE MODERN HARBOUR.

carded Alexander's Great Harbour, which had been for many years too difficult and shallow for shipping, and the entrance to which was still dangerous and difficult to make. They returned to the western side of Pharos and their great breakwater, like the south mole of the ancient harbour, was based on Ras el Tin. The modern harbour occupies what was the roadstead of its predecessor of 4,000 years ago.

TYRE.

Tyre was another famous pre-hellenic harbour (*Figs.* 7 and 8, pp. 12 and 13), but it is only a few years ago that a true plan of its works was published by Père A. Poidebard. As a result of 3 years' research, from 1934 to 1936, in which he made brilliant use of aerial observation and photography, coupled with submarine observation and photography, Poidebard was able to demonstrate the incorrectness of all previous plans and the unreliability of any plan made of ancient works unchecked by careful research and observation on the spot.

History has given the Phœnicians a reputation as builders and engineers. A delightful story is told by Herodotus in his description of the cutting of the Canal of Athos, which illustrates their skill as engineers. "When the trench grew deep," he writes, "the workmen at the bottom continued to dig, while others handed the earth, as it was dug out, to labourers placed higher up upon ladders, and these taking it, passed it on further, till it came at last to those at the top, who carried it off and emptied it away. All other nations, therefore, except the Phœnicians, had double labour; for the sides of the trench fell in continually, as could not but happen, since they made the width no greater at the top than it was required to be at the bottom. But the Phœnicians showed in this the skill which they are wont to exhibit in all their undertakings. For in the portion of the work which was allotted to them they began by making the trench at the top twice as wide as the prescribed measure, and then as they dug downwards approached the sides nearer and nearer together, so that when they reached the bottom their part of the work was of the same width as the rest." As builders they are, as everyone knows, renowned for the work they did for Solomon in building the temple of Jerusalem, whose "great stones," "wrought stones," and massive brass pillars 18 cubits high, modelled on those in the temple of Melkart, at Tyre, so impressed the Jews.

Tyre had two harbours (*Fig.* 8), the Sidonian on the north of the island and the Egyptian on the south, and like Pharos, a spacious roadstead to protect ships from the stress of the open sea when making the entrances. The Sidonian was what the ancients called a closed (*kleistos*) harbour; that is to say, it was within the circumvallation of the city and its entrance could be blocked by suspending a chain from one side to the other. The Egyptian was an open (*aneimenos*) harbour, outside the fortifications but adjoining them.

Tyre was a very old city, dating back, according to Herodotus, to

2750 B.C. This is probably incorrect, but at all events by 1400 B.C. it renown was widespread, and by 1100 B.C. its seamen had passed Gibralta and had dared the Atlantic. It was probably about this time that th Sidonian harbour was built. Hiram, king of Tyre (970–936 B.C.) friend an ally of Solomon, was a great builder and engineer. When he came to th throne Tyre was separated into three islands by arms of the sea full o reefs. Hiram filled these channels and on part of the land so reclaime

Fig. 7.

TYRE.
Note incorrect position of Egyptian harbour.

built the Egyptian harbour, not as Maspero and others have asserted on the south-east of the island (*Fig.* 7), but, as Père Poidebard's discoveries have shown, along its south coast (*Fig. 8*). A massive mole, 2,500 fee long, runs from the south-east corner to a large exposed rock lying off th south-west corner. Two similar moles, one running northwards from the rock at the end of the south mole, and the other running southward from the shore of the island, enclosed the harbour on the west. The end of these walls overlapped so as to form a protected entrance from th open sea to the western basin.

Two marked advances had occurred in constructional methods since the days when the harbour of Pharos was built, namely, the use of concrete in making sea-walls, and the use of iron dowels run in with lead. Both of these methods were used at Tyre.

The moles were very solid structures (*Figs. 9* (*a*) and (*b*), facing p. 14). They had foundations of large, hewn, rectangular blocks, all laid as headers. The middle was composed of hard concrete divided at intervals into com-

Fig. 8.

TYRE.
Showing correct position of Egyptian harbour.

artments by transverse bonding. The side bordering the sea was faced ith squared slabs, 10 feet long by $4\frac{1}{2}$ feet thick, laid as stretchers. The outh mole varied in width from 24 feet to 26 feet, whilst the two western ıoles, which had to face the full force of the sea, were $7\frac{1}{2}$ feet wider.

In the middle of the south mole was the main entrance to the harbour, ıd from each side of it two large wharfs, built of concrete and faced with one, were built across the interior for about two-thirds of its width. The arrow passage thus formed was commanded by a fortified post on the land. This passage was the boundary between the Western and Eastern asins.

A concrete wharf, the *Quai de la Source* on Père Poidebard's plan, cut th
eastern basin into two, the farther and smaller one of which appears t
have been paved throughout with flagstones and to have been used as
neorion, or shipbuilding and repairing yard, equipped with slips and store
houses. Père Poidebard thinks that it may have communicated with it
neighbouring basin by means of an inclined plane, but M. Bertou though
it had direct access to the sea. Possibly both methods existed. At th
northern corner of the outer eastern basin, where the *Quai de la Sourc*
abuts the island, was a small basin which accommodated a drinking-wate
tank for replenishing ships—an important item, for water was preciou
in Tyre, nearly the whole supply of the island having to be brought acros
by boat from springs on the mainland.

The Sidonian harbour made use of a small bay at the north-east sid
of the island and was partly surrounded by the city. Two jetties, on
jutting out from the ancient tower near the modern lighthouse and th
other coming from the opposite side in a northerly direction to meet it, pro
tected its entrance. Père Poidebard was able to trace the northern jetty
and thus to prove that it lay some distance beyond the existing jetty o
Sur and that the ancient harbour was larger than the modern. The con
struction of the jetties was similar to that of the moles in the Egyptia
harbour.

Old authorities record that the two harbours were connected by a cana
and many old plans show this canal, but it is not shown on Père Poide
bard's plan, or on Berthou's, made in 1846. It is, however, possible tha
there was communication through the arm of the sea said to have been re
claimed by Hiram. It was a common custom in ancient harbours to hav
two separate but interconnected basins, and Sidon, which also belonged t
the Phœnicians, was laid out on this plan, which had obvious advantage
Vessels could enter one of the basins when a contrary wind prevented the
from entering the other; if one basin was made unsafe by a storm, ship
could move through the canal and take refuge in its neighbour; whilst
an enemy attacked he would have to split up his fleet or risk being su
prised by the defenders who, having escaped through the other entranc
might attack him in the rear.

In addition to its harbours, Tyre took care to protect its roadstead
(*Fig. 10*, p. 15). North and south of the island ridges of rock, partly sub
merged and partly exposed, stretched parallel to the coast and formed
natural barrier against the waves. That they were not, however, considere
sufficiently effective has been made clear by Père Poidebard's discovery
traces of two separate lengths of wall based on the southern line of reefs, o
about 1,000 feet and the other 1,650 feet in length. These walls were
massive structure, 100 feet wide, and were faced with rocks, some of whic
were 10 feet square by 2½ feet thick and weighed about 15 tons (*Fig. 1*
facing p. 15). Probably, although sufficient evidence is not yet availabl
there was a similar reinforcement of the north reef. Traces seem still to hav
been in existence when Maundrell visited Tyre in 1697, for he reports th

Figs. 9.

(a)

(b)

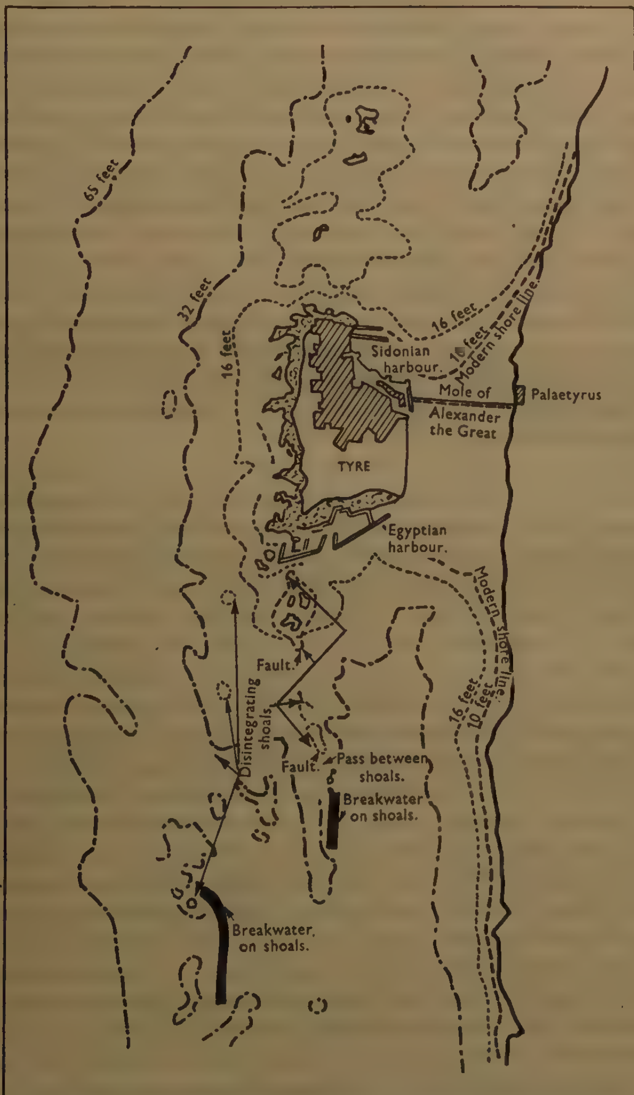
FOUNDATIONS OF THE MOLES OF THE SIDONIAN HARBOUR.

Pres. Address.

Fig. 11.

the harbours were, "in part defended from the ocean, each by a long ridge resembling a mole stretching directly out on both sides, from the head of

Fig. 10.



ROADSTEAD AND REMAINS OF BREAKWATER ON SHOALS, TYRE.

the island; but these ridges, whether they were walls or rocks, whether the work of art or of nature, I was too distant to discern." That they were in part works of art is proved by the fact that the stone used is

different from the rocks upon which it is laid, and that it must have con from quarries on the mainland where a similar stone is found. One cann help wishing that more information was available as to how these immen masses of stone were conveyed to the spot and laid with such accurac M. Henri Watier, whom Poidebard consulted, considered the constructio of such works perfectly practicable in antiquity. "Several divers could, he says, "easily push stones of nine tons weight into place as they we being let down by ropes." The divers of Tyre, who were accustomed t collect the shellfish *murex* for the famous purple dye, would be ideal for suc work. It is known that they could remain below water for 1½ minute.

Tyre enjoyed many centuries of fame as the finest and richest city i the world. All will recall the three vivid chapters in which the proph Ezekiel describes the city and foretells its fall—"thy riches, and thy ware thy merchandise, thy mariners and thy pilots, thy calkers, and the ex changers of thy merchandise, and all thy men of war, with all thy compan which is in the midst of thee, shall fall into the heart of the seas in the da of thy ruin." Even in the bitterness of his scorn he cannot refrain from note of admiration; "by thy wisdom and by thine understanding tho hast gotten these things."

Five hundred and eighty years after the death of Hiram came Alexande the Great. Tyre, unconquered still, was too great a danger to leav behind him while he was away subduing the East. Alexander's fleet wa too weak to fight her at sea. Nothing daunted, he attacked from the lanc and for this purpose he built a colossal mole 100 feet wide and ½ mile lon in 3 fathoms of water, so that Tyre ceased to be an island and became peninsula. He demolished the old city of Palætyrus for stone and robbe the forests of Lebanon for timber to accomplish his purpose. In 9 month he completed his task and captured the city. The laws of nature asserte themselves; coastal drift completed what Alexander began, and now Su the ancient Tyre, is connected to Syria by a broad neck of land.

The following interesting example will illustrate the efficiency of th ancient harbour engineer. Some years ago a friend of mine went out t advise on the construction of a harbour in the Black sea. After carefu study, he recommended a plan for a rubble stone breakwater protecting deep-water pier. On his return journey his ship called at Samsoun, th ancient colony of Amisus. As he had never been at Samsoun before, h went ashore, and was interested to find the ruins of a rubble breakwate sheltering a massive quay-wall, made of great blocks of masonry, whic might almost have been built to the plans he had just drawn up. Th ruins dated back to the days of Darius, say about 500 B.C. and I am ver tempted to see in them the "wisdom and understanding" of a Tyria engineer, for it is known that the Phœnician interests extended thus fa Perhaps there is a powerful *genius loci* in the Black sea; be that as i may, it is interesting that a Phœnician engineer (if my surmise is righ and a British engineer, separated in time by 2½ millenna, should hav solved a problem in almost exactly the same way.

Grecian Harbours.

When we come to Grecian times a rather different state of things is found. The shores of Greece and those of most of her colonies abounded in deep bays and long arms of the sea stretching inland, forming excellent natural harbours that required little in the way of artificial works to make them safe refuges. Moreover, Greece was divided into many small states, each of which, except Doris, Arcadia, and a few others with no seaboard, had its own port. Great harbours of cyclopic stonework like Pharos and Tyre were, therefore, unnecessary. Generally all that their natural harbours needed, apart from quays and wharfs, were short moles to narrow the entrance.

In the early days Athens used the broad open bay of Phalerum, where ships were beached in sight of the city. That arrangement had several disadvantages. In a surprise attack the enemy might land and paralyse the defenders before they could get down from the city and launch their ships; a more serious and permanent objection was that vessels had to lie out in the open exposed to the elements, an important fact when it is remembered that no voyages were undertaken between November and March. When the Persian danger arose, Themistocles, in 493 B.C., persuaded the Athenians to transfer their shipping to the fine natural harbour of Piræus and its two small neighbouring land-locked bays of Zea and Munychia (*Fig. 12*, p. 18). The works initiated by Themistocles and completed by Pericles gave Athens one of the safest and most convenient harbours in the ancient world. All three harbours were enclosed in one circuit of fortifications and connected to the city by the two famous long walls. The natural entrances to Piræus and Munychia were reduced in width to 55 yards and 40 yards respectively by the construction of solid breakwaters. Zea needed no narrowing. Apparently those breakwaters were constructed by heavy rubble thrown into the water and allowed to assume a natural slope. When the mound thus formed reached water-level a superstructure of huge blocks, some of them 10 feet square, fastened together with iron cramps, run in with molten lead, was built. This was the usual type of Grecian pier. Piræus, the main harbour, was divided into three chief basins, the mercantile harbour, in the centre, which occupied most of the area, the small corn harbour on the north, and the war harbour in the south. In the centre was the *agora*, or market, of Hoppodamus; on the western margin of the War Harbour (the *Kantharos*) extended the emporium or *deigma*, flanked by a series of porticos, the centre of commercial activities; near the entrance to the corn harbour was another large *agora*. Around the three harbours shipsteads were built, in which vessels could lie high and dry. They formed an essential part of the dockyard, especially for warships, which put to sea only on active service. If the triremes were left lying in the water they soon became leaky and unseaworthy, and also were liable to be attacked by the *teredo*. Their wooden fittings were stored alongside the vessels in the shipsteads; hanging tackle, sails, and ropes

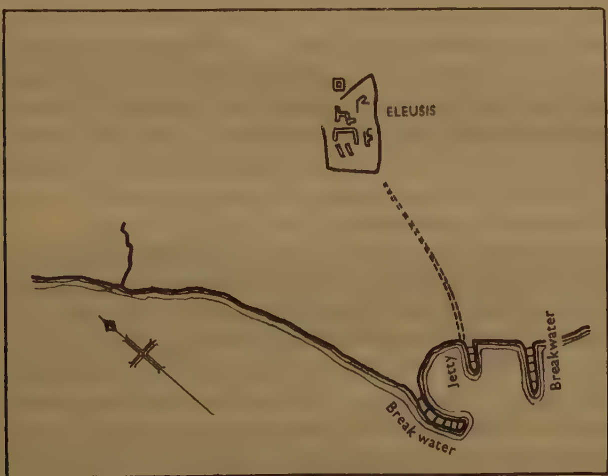
Fig. 12.

Plan of Piraeus, Zea and Munychia

were kept in the large arsenal at the entrance to the War Harbour. Traces of such buildings in Zea and Munychia are still in existence; those around Zea were roofed by low gables supported on stone columns, each gable sheltering two triremes.

Piræus, Zea, and Munychia were typical examples of the Greek natural harbours. At some places, however, artificial harbours had to be constructed, of which that at Eleusis (*Fig. 13*) may be regarded as typical, as the others were planned on a similar general principle. Two breakwaters were built out from the shore, curving inwards to form a narrow entrance between their ends, the space enclosed being an obvious imitation of a natural bay. Within the harbour was a jetty. This jetty and the break-

Fig. 13.



Eleusis Harbour.

water were constructed in the same way, with a foundation of dumped stone and a superstructure of large blocks held together by iron dowels. In all cases the material used was stone, probably because the art of pile-driving was not yet sufficiently developed to make the use of piles safe in harbour engineering, although piling had already been used in house-building for many centuries, and probably, also, because piles were liable to attack by *teredo*.

Roman Harbours.

"Italy," wrote Mr. H. Stuart Jones, "is not furnished by nature with many good harbours. The estuaries of her greater rivers—the Po and the Tiber—are subject to rapid accumulation of alluvial deposit, and some of her natural roadsteads, such as Antium, are rendered unfit for remunerative harbour-works by reason of their shifting sands. Few are the harbours

such as Brundusium, where a safe anchorage is provided by natural spits and promontories. The Romans were therefore obliged to face technical problems of no small difficulty when their growing commerce demanded effectual shelter in the ports of Italy." The Romans were essentially practical people, and in dealing with those technical problems they introduced many new methods, among which the most outstanding were the use of the arch, the cofferdam, hydraulic cement (pozzuolana) and the driving of piles in deep water. The discovery of pozzuolana in the 3rd century B.C. brought about a radical change in building and civil engineering structures. "Mixed with lime and rubble" wrote Vitruvius, "it not only furnishes strength to other buildings, but also when piers are built in the sea, they set under water and can be dissolved neither by the waves nor by the power of the water." The Egyptians, as I have shown, used the cyclopic dry-stone structure; the Greeks used large ashlar masonry held together by iron dowels and lead; the Romans used their famous, almost everlasting concrete made of pozzuolana, lime, and stone; and it was pozzuolana that rendered possible the erection of those gigantic vaulted structures found all over the Empire. Piles were used in bridge-work and foundations; but the great importance of pile-driving, so far as we are concerned at the moment, was that it enabled the engineer to make cofferdams for pier-building.

Vitruvius, in his treatise on architecture and civil engineering, *De Architectura*, written at the beginning of the 1st century A.D., has at the end of the fifth book a short chapter on harbour engineering. His object was to deal with the methods by which ships could be protected against storms and tempests. After a reference to the usefulness of natural harbours, he explains the technique of building breakwaters by means of cofferdams (*arcae*). In the last section of the chapter he states that shipyards should have a northern aspect whenever possible, because southern aspects, owing to their warmth, generate dry rot, *tinea*, *teredo*, and other kinds of noxious creatures. In any case, he says, wood should be used as little as possible on account of its inflammable nature. His remarks on the construction of breakwaters are of considerable interest. Four different methods are described. In the first case, where a masonry dam had to be made in the sea, he advised a cofferdam made of oak piles bound firmly together with chains. When this was finished the bottom was to be levelled and cleared, and a platform of beams laid upon it. The whole space above this was to be filled with stones embedded in a mortar composed of 2 parts of hydraulic cement to 1 part of lime. Next he discusses what should be done in places where hydraulic cement is unobtainable. In this case a double cofferdam should be built and the spaces between the walls of each cofferdam filled with clay in wicker baskets, tightly rammed down to make them watertight. The interior was then to be pumped dry by means of water-screws and water-wheels, and, if the bottom were hard ground, a concrete wall composed of stone, lime, and sand was to be built upon it, the lower portion being made wider than the upper. If, however (and this is his third method) the

ground at the bottom was soft, the foundation had to be prepared by putting down a layer of piles of charred alder and olive-wood filled in with charcoal. On this the outsides of the walls were built of squared stone, with the longest possible joints, so that the middle stones might be well tied together by the bedding. The middle was filled with rubble or masonry work. In a very difficult passage, he describes a fourth method, to be employed when it was not possible to use cofferdams owing to the violence of the sea. A mound was built out as far as possible, at the end of which small walls, springing from just below the water, were built up to the level of the top, forming an empty space between themselves and the slopes of the mound. This space was filled with sand, and formed what he called a margin. On this margin a large pillar of masonry was built and was left for 2 months to dry; after that period the walls were cut away, and when the sand was scoured by the action of the waves the pillar fell into the sea as a solid monolith. "In this way" says Vitruvius, "as often as is necessary, the pier is carried further into the water." It must, however, have been a very slow process.

The Roman ideal plan of a harbour is clearly expressed by Virgil in the first book of the "Æneid":

> "*Est in secessu longo locus: insula portum*
> *Efficit objectu laterum; quibus omnis ab alto*
> *Frangitur . . .*"

("There is a deep bay in a roadstead; an island forms it into a harbour by the shelter of its sides, which break every wave from the open sea.")

This, translated into an artificial harbour, presents us with the two incurving breakwaters of the Greeks, but with the Roman addition of a short protecting mole or island breakwater in front of the entrance, a type found in the important harbours of Antium, the Claudian harbour at Ostia, Centum Cellæ, etc. (*Fig. 14*, p. 22).

There were, however, exceptions to this rule. At Puteoli, on the bay of Naples, one mole originally protected the harbour. It was of a peculiar type introduced by the Romans, consisting of an arcade of fourteen arches resting on fifteen piers, each about 50 feet square. The foundations of the piers were built of pozzuolana concrete, as laid down by Vitruvius, the upper portions being filled with fragments of tufa and brick. In addition to the mole there are also remains of a number of basins protected from the sea by a double row of piers; those in the outer row were rectangular and probably carried arches, whilst the inner piers, opposite the open archways, are trapezoidal in section. Caligula built a floating bridge from the end of the main pier across the bay to Cumæ, a distance of 2–3 miles, which probably had also the military object of protecting the upper end of the bay of Naples against attack by sea.

The sand problem caused the Romans considerable trouble. Although some form of dredging is said to have been practised by the ancients in maintaining and deepening their irrigation channels, no record exists that it was ever developed sufficiently to enable them to use it to deal with

Fig. 14.


River Tiber.
Road
Aqueduct
Trajans canal
Quays and warehouses
TRAJAN'S HARBOUR
Inner harbour
HARBOUR OF CLAUDIUS
Arsenal
Market
Arched mole
Lighthouse
Solid mole
Mediterranean sea

silting in river-channels and harbours. The arcade form of breakwater was an attempt to use the tidal current to scour harbours, but usually failed in its purpose. The problem remained and silting drove the Romans from the harbour at Antium, and from the Tiber, and turned the magnificent harbour at Ostia into a failure. Speaking of the problem at the mouth of the Tiber, Sir John Rennie wrote, "Upon referring to the history of the shore, at the mouth of the Tiber we find that from the foundation of Ostia by Ancus Martius in 634 B.C. to the end of the Commonwealth in 82 B.C. the line of shore had advanced about 1,100 yards in 552 years; again from the Commonwealth to the end of the Empire in A.D. 364, a period of 446 years, it had advanced also about 1,100 yards, and from the Empire to the present time, being a period of about 1,400 years, it has advanced 2,550 yards, making a total distance of about 3 miles 600 yards in 2,480 years; and a projecting delta is formed at the mouth of the Tiber."

Many efforts were made to keep the Tiber open below Rome by revetting the banks and controlling the channel to induce scour, but all in vain. Gradually all shipping, except boats of the shallowest draught, was forced down to the lower part of the estuary, whence goods had to be transferred by barge to Rome. A great deal of the trade was transferred to Puteoli, which came to be regarded as the port of Rome and rose to the position of the premier commercial harbour of Italy; but its distance of about 140 miles from the metropolis, along the Via Appia, formed a serious inconvenience in view of the slowness of transport in those days. Moreover a safe harbour nearby was needed to accommodate the fleet which had the duty of guarding the mouth of the Tiber. Cæsar realized the urgency of the problem and proposed to build a new port, but he was prevented from doing so by the objections of his engineers. In A.D. 43 Claudius overruled these objections and gave orders to proceed with the work (*Fig. 14*, p. 22). A spot was chosen on the sea a short distance north of the river-mouth, and the place was called Ostia, after the town which had been the centre of the port works of the river harbour. This harbour had two basins. The outer was formed by two artificial moles, each 1,900 feet long and 180 feet wide. Both moles ran out almost at right angles from the shore for nearly half their length, and then curved inwards, leaving a space of 1,100 feet between their extremities. Immediately in the centre, and between the extremities, was an isolated mole, 780 feet long by 400 feet wide, leaving an entrance of 160 feet on either side. To form this mole the ship which had conveyed a huge obelisk from Alexandria to Rome for Caligula's Circus was filled with concrete and sunk. Great concrete masses were then piled on the top of it until the mole reached the surface. A lighthouse after the model of the famous Pharos of Alexandria was built on this island mole. The circular part of the main northern breakwater was constructed upon arches, in the hope that the current would prevent accumulations of sand The southern breakwater was solid throughout, to prevent the entrance of drifting silt and sand from the mouth of the Tiber. The depth of water

in the basin is unknown, but Sir John Rennie estimated that it would rang from 15 feet to 20 feet at low water. The area was about 130 acres. A the upper end of this main basin was a smaller one 1,200 feet long an 520 feet wide, covering an area of about 7 acres. It was separated fron the main basin by an island mole similar to that in the main entrance A very large portion of the harbour was dug from the mainland, and it i said that this involved the excavation of 80 million cubic feet of earth. I

Fig. 15.

CENTUM CELLAE HARBOUR.

spite of the vast amount of money and care expended on this work the harbour was not a success. Tacitus reports that 200 ships were sunk in the harbour itself during a storm in A.D. 62. Trajan (A.D. 92–117) added an inner basin, hexagonal in shape, with an area of about 70 acres. Claudius had dug two canals, running parallel to each other, connecting the harbour with the sea and the Tiber. To remedy this, Trajan took up part of one of these canals in creating his new basin and filled up the other. He then dug a fresh canal, which has since become the mouth of the Tiber, the river having deserted its old course. The harbour was well provided

vith quays, transit-sheds, and store-houses, some of which were finished, egardless of expense, with marble tiling.

The Roman engineers were right when they advised against building his harbour. The forces of nature were against it from the beginning, and o-day the remains of the great port of Ostia lie buried in sand a mile from he shore. The tendency must have begun to become obvious even in the eign of Trajan, for he took measures to provide a new harbour for Rome little higher up the coast. The result was the harbour which, under its aodern name of Civita Vecchia, is now the principal port of Rome. Centum 'ellae (*Fig. 15*), to give it the name by which it was then called, was lanned and built on precisely the same principles as those employed

Fig. 16.

CELLAE AT CENTUM CELLAE.

t Ostia, except that in it the island mole overlapped the ends of the main reakwater, instead of lying between them. The harbour, as its name mplies, was provided with one hundred covered *cellae*, or docks for warhips (*Fig. 16*).

Pliny the Younger, nephew of the naturalist, when staying with Trajan in the locality, visited the new port while construction was going on, and wrote a description of it in a letter to his friend Cornelianus.

"The house is most beautiful," he wrote; "it is surrounded by green fields and overlooks a bay where, at this very moment, a harbour is being built. The breakwater on the left side is already finished and is a work of great solidity. The one on the right is still under construction. In front of the entrance to the harbour an island is being formed, which by opposing the storm breaks the force of the waves and forms a safe passage for ships on each side. The construction of this island is a work of art that is well worth seeing. Enormous blocks of stone are brought in great barges and tipped, one on top of the other, into the water. Their immense weight and mass keep them steady and gradually they heap up and form an embankment.

Already a ridge of rocks, which breaks the driven waves and throw them skywards in a cloud of spray, is beginning to appear abov sea level. The crash of the foaming sea is tremendous. Piers wi afterwards be built on the rocks and in the course of time th impression will be that of a natural island rising from the water This port will be named after its maker, indeed it has already bee so named, and it will save, one may say, a multitude of lives; fo this coast, which for a long stretch is without a harbour, will no have this one as a refuge for ships."

The Roman Empire was followed by a period of more than a thousan years of quiescence, or even retrograde action, in harbour engineerin I know of no great harbours, such as those which I have described, tha were built during the dark periods of the Middle Ages. We have to wa till the great engineering revival that began about the middle of the 18t century before we find such ambitious schemes again attempted.

It is, however, interesting to study the debt we owe to the ancient The similarity of their treatment of problems to the methods of the moder engineer is, as I have tried to show, in many cases very remarkable. have touched only the fringe of the subject, but that has been sufficie to convince me that it is one well worth deeper study and research.

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Paper No. 5237.

"Beach Formation by Waves; Some Model-Experiments in a Wave Tank."

By MAJOR RALPH ALGER BAGNOLD, M.A.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

THE USE OF MODELS FOR RESEARCH ON BEACH FORMATION.

'HE shaping of loose bed-material into beaches by the action of water 'aves has been studied in nature by geomorphologists and others, but, wing to the difficulties involved, only scanty information is available bout what happens beneath the water-surface, and about the factors 'hich control the shape of the mature beach profile. These difficulties re obvious. For instance, the lower portion of the beach cannot be seen uring a storm; natural waves are necessarily uncontrolled, and are ldom constant in character for any length of time; and the whole rocess of beach formation is complicated by tidal variations, both in the ean water-level and in the direction and intensity of coastwise currents.

An investigation with models, on the other hand, is hindered by none f these difficulties; the movement of the water and of the material can e readily watched and measured over all portions of the beach, under e action of any required type of wave, repeated at will. Moreover the

[1] Correspondence on this Paper can be accepted until the 15th March, 1941, and ll be published in the Institution Journal for October 1941.—SEC. INST. C.E.

influence of the particle size on the beach profile can be demonstrated directly by a change of beach material while the wave characteristics are kept constant.

It must be remembered, however, that model-experiments are subject to the following limitations:

(*a*) In a narrow wave-tank the movement, both of the water and of the beach material, is constrained to two dimensions, the horizontal movement being forward and backward only. But on an open beach there may be important coastwise components as well. These may be of two kinds: (i) a steady transverse current, which, although usually far too weak itself to set any bed-material in motion, adds a continuous coastwise drift to the oscillatory bed-movement due to the waves. It is this drift which ultimately controls at any given spot the supply or removal of beach-building material. Although it cannot be reproduced in the wave tank, its effect on the availability of beach-building material can nevertheless be imitated by the direct addition of material to, or by its removal from, the tank during the progress of an experiment. (ii) Even though there is no coastwise drift, and though the direction of wave-advance is at right-angles to the beach, the movement of the surge over the beach may deviate from this direction and take the form of a sideways sweep. Such a deviation is particularly noticeable on gently-sloping sand beaches; it cannot be imitated in the tank. Its effect is discussed later in this Paper.

(*b*) Before the results of model-research can be applied with any certainty to full-scale phenomena, a sufficiently sound picture of the physics must be obtained, so that reliable model rules can be formulated. It is clear that research in this case must be largely empirical, since no mathematical theory at present underlies the physics either of the breaking wave or of the carriage of solids over a bed by a steady stream, let alone their carriage by an oscillating stream with powerful accelerations. Hence model rules, that is to say, the definition of the conditions governing similarity of beach profile for differing linear dimensions, can be but approximate and can only emerge as the research proceeds.

It may be useful to make an initial assumption regarding the similarity of beach profiles, in order to determine the scale of the model to a first approximation, and later to examine the general validity of this assumption in the light of the experimental results. It will be assumed, therefore, that beach profiles should be similar if the ratio:

$$R = \frac{\text{amplitude of the wave : } h}{\text{diameter of the beach particles : } d}$$

is constant.

The Scope of the Experiments.

Experiments by the Author on the shock-pressures exerted against a vertical sea-wall by breaking waves have been described in a previous

Paper[1]. Full details of the wave-tank used (Fig. 1, Plate 1) were given therein. The tank was 110 centimetres high and 53 centimetres wide in cross-section, and was provided with glass sides. A 10-centimetre grid was fixed to one side for easy measurement, as shown in Figs. 3, 4, 5, *et seq.* (Plate 1). The left-hand edge of the glass coincided with a vertical concrete wall. The waves, which were created by a hydraulically-operated paddle, advanced from right to left, and were made to break by the distortion imposed upon them by the presence of a sloping beach. This beach consisted of a rigid steel plate abutting on to the wall and inclined at an arbitrary angle.

A further series of experiments, upon which this Paper is based, were planned with the object of discovering what the ultimate beach profile would be if the steel plate were to be replaced by loose shingle or sand, and the waves themselves were allowed to fashion their own beach. These experiments were in progress at the outbreak of war, and were interrupted by the departure of the Author on military service abroad. The results, although incomplete, may be worth recording.

An idea of the range of model-scales upon which the experiments were conducted may be obtained from the following figures. Assuming 3 metres to be the amplitude of an average real storm-wave, and a range of possible sizes of real bed-material between 8 centimetres and 0·05 centimetre, the values of R (as defined on p. 28), which are found in nature, range from 37 (shingle) to 6,000 (sand). In the model the waves varied in amplitude between 30 centimetres and 5 centimetres, and the size of the material varied between 0·7 centimetres and 0·05 centimetres. The values of R_m in the model therefore covered a range between 7·5 and 600. It will thus be seen that the whole range of natural shingle and pebble conditions was more than covered by the model, but that the model failed to imitate the extreme natural case of large waves and fine sand. It was intended to investigate this case later by the use of fine pit-sand of 0·007 centimetre diameter, which would have given values of R up to 4,300.

Terminology.

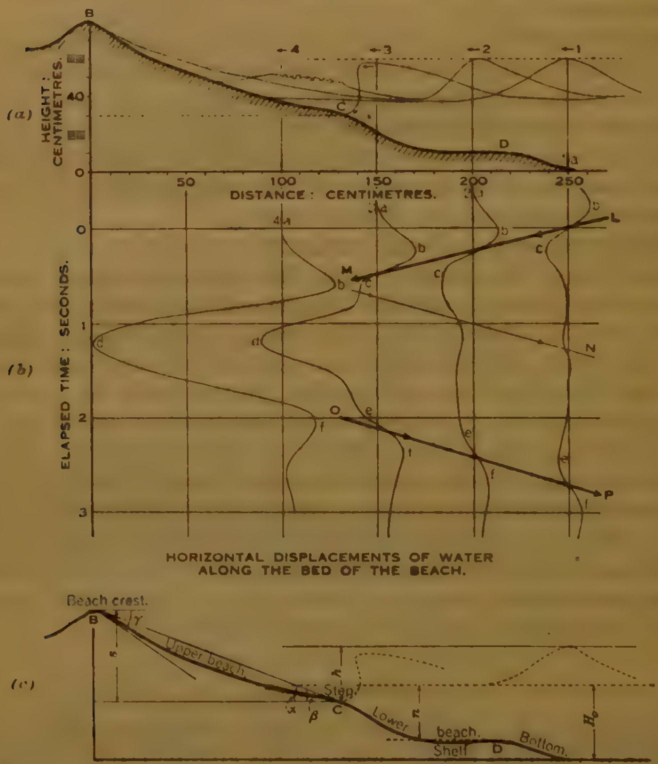
Figs. 2 (*a*) and *2* (*c*), p. 30, show the profile of a typical shingle beach as thrown up in the wave tank by waves of the kind indicated. Though the following terms have special reference to such a beach, they can be applied with but slight modification to beaches formed under different conditions.

The Wave-Amplitude (h). The total amplitude, as measured by the difference of level between the crest and the trough of the wave " out at sea ", bears no known relation to the dimensions of the distorted wave near the beach. Moreover, owing to the limited length of the experimental tank and to the fact that the wave is never symmetrical about any

[1] Major R. A. Bagnold, " Interim Report on Wave-Pressure Research. Journal Inst. C.E., vol. 12, p. 202 (June 1939).

horizontal plane, the " mean level " is difficult to define and is clearly not the same as the " still-water level " (H_0). The wave amplitude has therefore been defined, for the purposes of this Paper, as the difference between the crest level " out at sea " and the level of the lowest exposed beach-line immediately in front of the advancing wave. As thus defined, the amplitude is independent of wave-length, wave-shape, and still-water height. Both the levels involved are easily measured. The level of the

Figs. 2.



lowest exposed beach-line will also be taken as the datum level for measurements of the height to which both the surge and the beach crest rise. This datum level happens to coincide very closely with that of the " step."

The surge height (s). This is the greatest height above the datum leve to which the water surges up the beach.

The beach crest. Whenever a new beach is thrown up (provided adequat material is available) by a given train of identical waves, the height of th crest line B (*Fig.* 2(c)) coincides with the surge height s.

The beach. This consists of two concave zones BC and CD, which will b

called the "upper beach" and the "lower beach" respectively. They are separated by the convex "step" C. The centre of the step is found to coincide with the datum level of the lowest limit of the uncovered beach at the moment when the wave is in position 3 (*Fig. 2* (*a*)). The bottom portion of the lower beach is often horizontal, and may even terminate in an actual rise at D. This horizontal or nearly horizontal portion will be called the "shelf."

The shelf depth (n). This is the mean depth of the shelf below mean water-level during wave-action. For simplicity this mean water-level will be taken as approximately the same as the still-water level H_0.

The bottom. This refers to all the submerged surface to seaward of the limit D of beach movement.

The beach angle (α). This is the angle made by the line BC with the horizontal.

The upper-step angle (β). This is the slope of the flattest portion of the beach surface immediately above the step.

The crest angle (γ). This is the angle of the steepest beach slope immediately below the beach crest.

The Movement of the Water (Considering the Lower and Upper Beaches Separately).

The velocity c of advance of a surface-water wave depends in general on the wave-length λ and on the depth H_0 of the water, according to the equation:

$$c^2 = \frac{g\lambda}{2\pi} \tanh \frac{2\pi H_0}{\lambda}.$$

If the wave advances over a very gently rising bottom, till the depth becomes less than a quarter of the wave-length, the effect of the wave-length on the wave-velocity disappears. In the above expression $\tanh \frac{2\pi H_0}{\lambda}$ now approaches $\frac{2\pi H_0}{\lambda}$, so that λ cancels out, leaving:

$$c^2 = gH_0, \text{ or } c = \sqrt{gH_0}.$$

Under such conditions the wave is called a "long wave." The actual wave-length decreases as the wave continues to advance, because the foremost waves are travelling more slowly in the shallower water and tend to be overtaken by those behind, which are still in deeper water. When the wave-length is about 7 times the amplitude the wave becomes unstable and breaks. If, however, the sea bottom is bordered by a comparatively steep shingle beach, the wave's velocity cannot slow down sufficiently fast to keep pace with the rapidly decreasing depth; the equation, $c = \sqrt{gH_0}$, no longer holds good, and the fall in wave-velocity between the beginning of the beach and its extreme crest may be negligible.

The wave suffers one of two possible fates. (1) If its wave-length/ampli tude ratio was previously approaching 7 the wave is broken by the sudde distortion caused by the slope of the beach. *Fig. 2* (*a*) shows succeedin stages of the distortion. In the final stage the wave flattens out after th break and surges up the beach. (2) If the wave-length/amplitude rati was previously low the wave passes through the stage of maximun instability, when its forward face is raised to the steepest angle, and doe not break. It rights itself by pushing out the foot of its forward face at higher velocity than that at which its crest is moving. The wave there after flattens out into a beach surge, as in the former case. The mai difference between the two cases is that in (1) the foremost water of th surge up the beach has come from the overturned crest, whereas in (2) i has come from the foot of the wave face. These two cases are but extreme of a complete range of possibilities corresponding to a total break, variou degrees of partial break, and no break at all.

The experiments showed that on a shingle beach it makes no appreci able difference to the ultimate profile whether the wave breaks or not.

In *Fig. 2* (*b*) an attempt has been made to indicate, by means of dis placement/time curves, the observed movement, along the bed, of wate particles which were originally at rest in positions 1a, 2a, 3a, and 4a (al displacements along the inclined bed being represented as horizontal i the diagram). Comparative bed-water velocities are indicated, quali tatively only, by the relative flatness of the curves along the displacemen axis, and the accelerations and retardations by the relative abruptness o their curvatures. The advance and recession of the wave proper in th water above is accompanied by a wave of movement along the bed, an the propagation of this forwards and backwards is indicated by the line LM and OP. These lines are not continued to the ultimate beach crest because the surge up and down the beach can hardly be considered as travelling wave whose rate of propagation, as such, has any real meaning

Before proceeding to discuss the experimental results in detail, th following important general features should be noted :

(i) The drag on the bed-surface, which presumably controls the move ment of the bed-material, is known, from existing work on the carriage o solids by streams, to be some direct function of the velocity, the accelera tion, and the degree of turbulence in the lowest water-layers.

(ii) Owing to inevitable losses due to bed friction and similar cause during the beach surge, to the breaking of the wave, and to the partia reflexion from the neighbourhood of the step (shown by the line MN i *Fig. 2* (*b*)), the energy of the main outward wave OP is considerably les than that of the approaching wave LM. Hence the outward velocities an accelerations of the water are also less.

(iii) As a result of (i) and (ii), it follows that, of those particles (shingle which are so large that they are moved only during the most violent par of the cycle (the portions bc and ef of the displacement curves), a greate

number are set in motion by the inward than by the outward waves. Such particles therefore creep beachward, and the bigger the particle the more pronounced is the creep. The biggest mobile particles can only be moved beachwards, and therefore collect highest up the beach, along the crest.

(iv) On all steeply sloping beaches such as those dealt with here, whether or not they are composed of movable material, there exists a certain critical contour-line, above which the advancing wave-front is most vertically inclined. On mobile beaches the material at this level becomes fashioned into the step; but if the beach surface is rigid, this level still coincides with a critical position in the history of the wave. At this position the shallow reverse current 4ab (*Fig. 2* (*b*)) is so delayed by the drag of the beach close under it that it collides at M with the advancing current 3bc. As a result, a partial node or *clapotis* occurs, and the bulk of the water at this node is forced upwards, as indicated by the small arrows in the top figure. At the bed, both currents are momentarily halted, and a small secondary wave MN is sent out into deeper water. On the beachward side the surge may similarly be regarded as a secondary wave, albeit of far greater energy, reflected forward from the node. On its return from the beach crest the surge collides at the same place with the now almost stationary water, and again forms a *clapotis*. It is at this collision, at O, that the main return wave OP is generated.

The above picture of the water motion gives some physical background to the conception of two separate beach zones, the lower beach and the upper beach. The profile of each appears to depend on a different set of factors and to obey different model-rules. It will be shown that, whilst the character of the lower beach seems to depend only on the value of the ratio R of wave amplitude to particle size, that of the upper beach depends on the absolute value of each of these quantities independently, the height of the upper beach being proportional to the wave amplitude h, and its angle α depending on the porosity of the material, which is itself a function of the grain diameter.

The Initial Beach Profile: Outline of the Main Series of Experiments.

The main series of experiments was made with rounded beach pebbles of diameters ranging from 0·5 centimetre to 0·9 centimetre, with a mean diameter of 0·7 centimetre. In subsequent experiments sufficient work was done with other material, of diameter ranging from 0·3 centimetre to 0·05 centimetre, to ascertain how far the results were modified by changes in the particle size.

The standard initial beach profile was taken as that left by wave action during a receding tide. To produce this initial beach, material was

heaped steeply against the back wall, as shown in Fig. 3, Plate 1. The tank was then filled with water to a suitable level, and the wave-making apparatus was put into operation. When a beach of maximum height had been built up, the water in the tank was let out slowly, the wave action being maintained to give the effect of a falling tide. Apart from minor irregularities (due to certain resonant states being set up between the beach and the paddle) the resulting beach had a remarkably straight profile. Fig. 4, Plate 1, shows this low-tide beach for the 0·7-centimetre material, and Fig. 5, Plate 1, for the 0·3-centimetre material. In both cases the inclination of the beach was 19 degrees. (The grid in these and all subsequent photographs is divided into 10-centimetre squares.)

The tank was filled up again to a still-water level of 20 centimetres. Waves of very small amplitude were made to strike the beach. The amplitude was increased until the beach material began to move, and then it was kept constant, the wave action being continued till the resulting tiny beach became mature and stable. The wave amplitude was measured, and a photographic record was taken of the final beach profile.

With the water still at the same level, another series of waves of greater amplitude was run, until the new and larger beach became mature. The process was repeated with bigger and bigger waves up to the maximum amplitude possible for the particular water-depth used. The beach was then re-formed to the same initial profile, and the whole procedure was repeated in turn for a series of water depths ranging up to 50 centimetres. A selection of beach profiles for the 30-centimetre and 50-centimetre levels is shown in Figs. 6 and 7, Plate 1.

Formation of the Lower Beach.

For values of R below 45 (*large shingle*).

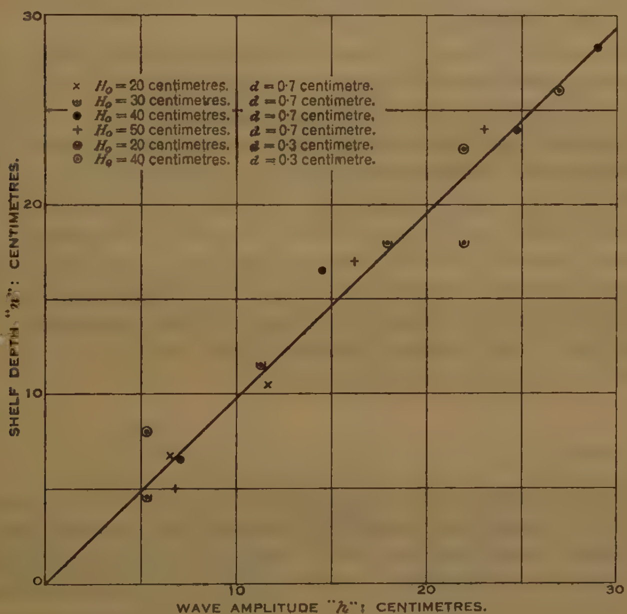
It will be seen from *Fig. 2* (*b*) that the initial acceleration at b, and the maximum value attained by the bed velocity of the water under the advancing wave, both increase as the wave passes over the rising bottom. At some point D (*Fig. 2* (*a*)) the violence of the bottom flow becomes sufficient to move a few of the most exposed pebbles forward up the beach. Farther on this motion is augmented by the addition of still more moving pebbles from the bed. The pebbles are carried to the neighbourhood of the step C, where their motion suffers a check (due to the momentary gathering-up of the water), before they are passed on to the upper beach. The returning wave ef causes a slight reversal of the pebble motion, but the net effect is a removal of pebbles from the whole zone between D and C. As the slope of the resulting shelf approaches the horizontal, fewer and fewer pebbles are moved forward by each wave, and the reverse motion

ceases altogether. The final stages of the excavation are very slow indeed. Ultimately all pebble motion on the lower beach ceases.

That the limiting depth at which pebble motion ceases can be slightly greater on the beachward side of D than at D itself is, no doubt, due to the progressive development of turbulence in the water as it passes over a greater length of beach surface, and to the consequently greater drag which the same mean velocity can exert on individual pebbles.

The width of the shelf, according to experiments in which the initial beach inclination was varied, depends on the initial profile. The flatter

Fig. 9.



this is made, the wider is the shelf. In beaches formed by the breakdown of an initially very steep shingle bank, there is no shelf or lower beach at all, the "made ground" of the step falling directly to the bottom at the angle of repose. (Fig. 8, Plate 1.)

The shelf-depth n below the mean water-level H_0 was found to be independent of H_0 and also of the grain-size d (at all events between the limits $H_0 = 20$ to 50 centimetres, $d = 0{\cdot}3$ to $0{\cdot}7$ centimetre, and $R = 16$ to 90). Though the shape of the oncoming wave and the profile of the bottom seem to have some minor effect on it, n was found to be approximately equal to the wave-amplitude h over the whole of this range. The experimental evidence is shown graphically in *Fig. 9*. Further experi-

ments with bigger grains are required to verify this relation for very small values of R.

For values of R above 45 (*Oscillation Ripples*).

That the depth to which the material is removed does not increase as the grain size is made smaller is a rather surprising result. The explanation appears to lie in the progressive changes which take place in the small-scale profile of the shelf surface and their effect on the water-flow near it. The flat shelf surface becomes unstable when R exceeds 45. Transverse ripples appear on it. These evidently exert an increasingly powerful drag on the movement of the lowest water-layers as the grain-size is made smaller and the grains are more mobile.

Figs. 10 and 11, Plate 1, show the effect of reducing the grain-size while keeping both H_0 and h nearly constant. In Fig. 10 the grain-size d was 0·7 centimetre, and h was 29 centimetres ($R = 41$). In Fig. 11, d was 0·3 centimetre and h was nearly the same at 27 centimetres ($R = 90$). It will be seen that in the latter case a single large ripple has appeared at E between the step C and the point D. Fig. 12, Plate 1, shows the further development of the ripple system as the value of R is increased. In this instance the grain-size was 0·05 centimetre, and h was 11 centimetres ($R = 220$). Here the concavity of the lower beach has entirely gone, and the beach has begun to assume the typical rippled appearance of a low-tide sand floor. The formation of these ripples and the movement of the water over them is dealt with on p. 50.

For large values of R.

As already mentioned, the experiments did not extend to cover those values of R corresponding in nature to the case of fine sand and large waves. Classical wave theory, however, rather indicates that at some high value of R a change should take place in the motion of the grains over the lower beach, for it has been shown that the advance of water-waves of large amplitude/wave-length ratio must be accompanied by a slow advance on the part of the upper layers of the water itself. This means that underneath there must be a corresponding drift of water outwards down the lower beach. The velocity of this drift is very feeble compared with the alternating water-velocities constituting the wave itself ; but its effect on particles so small that they are kept in almost permanent suspension over the rippled surface may well be a net outward drift of material instead of the inward drift which certainly prevails for large particles. The material removed may be expected to come to rest in such deeper water that the bottom motion due to the waves is not sufficient to pick individual particles of the bed.

Although this outward drift has not been observed in the model-tank

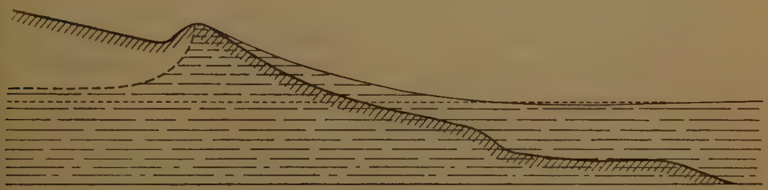
evidence for it seems to be provided in nature by the very different effects of big waves on shingle and on sand beaches. For whilst, under the action of gentle seas, sand tends to drift shoreward like shingle, a storm (R very large) tends to carry the sand out to sea. The critical value of R at which the change from shingle to sand takes place is probably of the order of 2,000.

The Upper Beach: Behaviour when Beach Formation is Unimpeded.

Movement over the Beach.

The water above the step is violently accelerated, and surges up the beach. It drags with it not only material from the step and from the shoreward side of it, but also all the new material which may have been passed up from the lower beach. The velocity of the surge is subsequently checked by the action of gravity. It becomes zero instantaneously over the whole length of the surge at the moment when the foremost water has reached the maximum height s above the step-level. There then remains, however, an appreciable water-velocity perpendicular to the surface, for the water continues to flow inwards and downwards through the interstices between the grains. The mass of water taking part in the return surge is therefore considerably less than that in the up surge. There are, in addition, energy losses due to friction and turbulence which also reduce

Fig. 13.



the velocity and the drag of the returning water. Hence, were it not for the inclination of the beach surface, which favours the downward movement of the material, more material would always be driven up the beach than was brought back again. Evidently once the beach is mature its surface assumes such an inclination and such a curvature that a state of equilibrium is everywhere set up, and just as much material returns as is driven up.

The water of the returning surge is violently checked as each element of it successively reaches, and piles up against, the stationary water above the step. The descending shingle comes to rest in a heap at the step by tumbling over its brink. The step therefore advances seawards by building out a steep slip-face inclined at the angle of repose.

The water which percolates into the shingle forms a water-table which, at the moment of maximum surge-height, has a cross-section as shown in *Fig. 13*, p. 37. The mean water-level within the shingle is maintained during the whole period of wave action at an appreciable height above the still-water level. Hence, except during the period of rising surge, there is an outflow of water through the submerged beach surface. This outflow is greatest during and immediately after the fall of the surge, when the shingle vertically beneath the crest is still nearly full, and the head of water is consequently high.

The Beach Angle α.

The beach angle was found to be entirely independent both of the wave-amplitude and of the water-depth over the whole of the experimental range. It appears to depend only on the absolute size of the grains composing the beach. The figures were as follows :—

Mean grain diameter : centimetre.	Beach angle : degrees.
0·7	22
0·3	19·5
0·05	14

The beach angle was remarkably constant for all the beaches thrown up with each kind of material, the variation being less than 4 per cent.

The Surge or Beach-Crest Height : s.

The final height of the mature built-up beach crest above the step was found to be proportional, for any given size of beach material, to the amplitude h of the " out-at-sea " wave above the same datum-level, so that :

$$s = bh$$

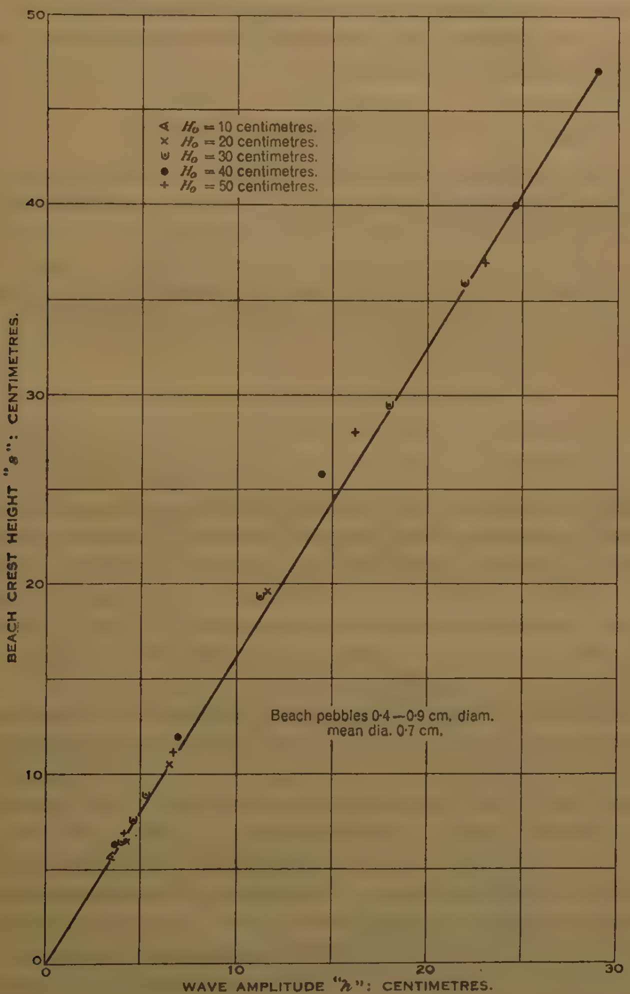
where b had the value 1·68 for the 0·7-centimetre material, and 1·78 for that of 0·3 centimetre diameter. Time did not permit of many readings being taken for the 0·05-centimetre material, but there is some evidence that for this b lies in the neighbourhood of 1·8. The experimental values of s in terms of h are shown graphically in *Fig. 14*. Once again the values are independent of the water depth H_0.

From these results, combined with those given under the heading " The Beach Angle ", it seems possible to predict the maximum dimensions of any given storm beach from the amplitude of the waves and the size of the beach material. Conversely, under well-chosen conditions, the height of the highest storm beach known to have been formed during modern times should give useful information as to the maximum wave-amplitudes likely to be experienced on that particular stretch of coastline.

In cases where there was no newly built-up beach, for example where

the value of h was too small for any grain movement to take place, or in the case of a breakdown beach such as that shown in Fig. 8, Plate 1, the surge still rises to the height given above.

Fig. 14.



Lateral Stability of the Beach Contours.

In all the experiments where shingle was used, the model-beaches which were thrown up by the unimpeded action of the waves set themselves at right-angles to the line of the wave-advance. When more

material was added on one side of the tank, this soon became distributed evenly over the width of the mobile portion of the beach. From this it appears that if waves strike a stretch of shingle beach obliquely, the beach-line will, provided that coastwise losses of material away from the farther boundary of the stretch are prevented, become re-oriented so that its contours run at right-angles to the line of advance of the waves. The conditions thus postulated are instanced in the accumulation of shingle in the corner cut off by a high groyne.

Continued supply of Material and Constancy of Profile.

A further experiment was made by adding a small but continuous supply of fresh material to the shelf after a mature beach had been thrown up. This was intended to imitate the accumulation of material by coast-wise drift. The effect was that the whole beach advanced seawards, but its profile remained entirely unchanged. The topmost surface, previously a narrow ridge, became a flat horizontal table as the beach top moved forward.

The Effect of the Presence of Impervious Material at the Crest, Lateral Instability, and the Breakdown of the Upper Beach.

As previously described, the original mass of beach material in the tank was heaped against a vertical concrete back-wall. In the course of the experiments, the wave-amplitude was gradually increased during the passage of a long train of waves until the water surging up the rising beach reached the neighbourhood of the exposed wall. That is to say, the vertical plane through the beach crest approached the plane of the wall.

It was found that the crest-line was now no longer stable and horizontal. Either one side of it or the other fell rapidly below the surge-level, and the surplus material accumulated on the opposite side. At the same time the surge-height up the wall on the denuded side rose from 1·68 h (for 0·7-centimetre material) to nearly 2 h, whilst that on the opposite side remained the same as before. A further slight increase in the wave-amplitude led to a breakdown of the whole beach. The crest-line became stable and horizontal once more, but at a considerably lower level, and was now wholly submerged by each wave. The surplus material went to extend the step to seaward (Fig. 15, Plate 1). The beach angle α made by the line joining the beach crest to the step had fallen from 22 degrees to 14 degrees.

The presence of the wall had caused a breakdown of the beach to occur before the free water of the surge had been allowed to reach such a height that it actually touched the wall above the shingle. Moreover, careful observation showed that the breakdown was not due to any slipping of the shingle down the wall, or to any internal collapse of the material. Clearly,

therefore, it can only have been due to the effect of the wall in preventing the free percolation of water through the material. To test this, an impervious steel plate (the top of which had been roughened by sticking beach pebbles to it) was inserted just below the sloping beach surface (Fig. 16, Plate 1). If this plate had been absent the beach material would have piled up till the crest angle γ reached the angle of repose—33 degrees—and the overall beach angle α had reached 22 degrees (for the 0·7-centimetre shingle used). The plate was, in fact, found to act in just the same way as the vertical wall had done. Although, again, no internal slipping was detected along the plate-surface and no part of this surface was initially exposed to the free action of the surge, yet the thin layer of material above the plate, instead of increasing in thickness, at once began to be removed by the surge. At the same time lateral instability was again noticed. The plate became denuded at random on one side or the other, and the material began to pile up on the opposite side. When, however, the surge height was slightly raised so that the surge reached the exposed plate-surface on both sides, stability again obtained. The now submerged beach-top made an angle α with the step whose value was exactly the same as that with the vertical wall, that is to say, 14 degrees.

As already stated on p. 38 this 14-degree beach-angle was that attained by the mature unobstructed beach composed of the 0·05-centimetre material (Fig. 12, Plate 1). Here again, but this time owing to the relative fineness of the interstices between the grains, the porosity of the material was low.

It seems, therefore, from the experimental evidence, that if no water sank into the material the beach angle would remain approximately constant at 14 degrees for a wide range of grain-size and wave-amplitude (for all values of R up to 400 at least). It should be noted that this statement is subject to the limitation, imposed by the wave tank, that the motion of the water is in two dimensions only, lateral circulation being negligible. This proviso appears to be fulfilled in nature in the case of shingle beaches of high porosity. The effect on the beach angle of the lateral sweeping movement of the surge over a very flat sand beach will be discussed later (p. 45).

Factors governing the Upper-Beach Profile. The Effects of Energy Losses and of Percolation (two-dimensional water-movement only).

The following ideas as to the behaviour of the upper beach are suggested by the experimental facts.

(1) The surge may be regarded as a special case of a *clapotis*, or standing wave, in process of being reflected off an unyielding wall. The case of the vertical wall has been investigated by Sainflou[1], who found that the surge-

[1] G. Sainflou, "Essai sur les Digues Maritimes Verticales." *Annales des Ponts et Chaussées*, vol. 98-*ii*, 1928, p. 5.

heights should be equal to $2h$ in the case of deep water, and slightly less for water of limited depth.

(2) If there are no losses of water energy by friction, internal or external, and no mass loss by percolation through the beach surface during the surge cycle, then the motion of the surge should be reversible, and as much material would always be carried back as was carried forward. Hence there would be no beachward drift of material and the beach angle would be zero. The surge would rise to the height calculated by Sainflou for a vertical wall.

(3) If there are such losses, then the mean drag force of the surge on the surface material is greater on the upstroke than on the downstroke. The excess force is just balanced by the component of gravity in the direction of the now inclined beach.

The surface angle of the beach at any given contour would, therefore, seem to depend on the ratio of the energy lost or dissipated by the surge over all the beach surface above that contour to the total energy which passed the contour on the upstroke. If this ratio is high, the relatively feeble downward drag of the returning surge cannot carry back a quantity of material equal to that previously brought up, unless the angle is steep and the assisting gravity component large.

The energy losses may be divided into two classes :

(*a*) A general dissipation of kinetic energy by friction against the bottom, and a dissipation of the eddy-energy of turbulence by internal fluid-friction ;

(*b*) Potential energy carried away by the water which percolates into the beach pebbles and does not take part in the return surge.

In the case of fine beach material the porosity is small and the energy-loss (*b*) by percolation is negligible, so that the inclination of the surface at every level up the beach should be the result of (*a*) alone. Experiment (Fig. 12, Plate 1) appears to show that the inclination under these conditions is constant ; the beach is flat and inclined at 14 degrees.

With material of larger grain-size the loss (*b*) by percolation is appreciable. Not only is the overall beach angle α increased, but since the proportional loss of energy by the disappearance of water through the surface is not constant from contour to contour, the surface is no longer plane. The proportional loss is greatest at the top, where the mass of the surge-water is nearly zero and where its potential energy is greatest. Here, in the case of pebbles, the entire flow of the water is into, instead of parallel to, the surface, so that all the pebbles which have been thrown up the crest must stay there. The top of the beach therefore piles up until the surface angle reaches the limiting value of 33 degrees. At this angle the pebbles are brought down by gravity alone.

Farther down the slope the proportional loss through the surface is smaller, and so the surface angle decreases. At a certain level the net

inflow through the surface must be zero. This level is presumably marked by the contour at which the surface inclination is 14 degrees. Lower still there is, as has already been pointed out, an outflow of water; so that the loss of energy by percolation is negative. The outward pressure against the surface pebbles here assists the downward movement of the material, and the surface inclination is, in consequence, less than that of a non-porous beach. The inclination of the beach close to the step (the upper step angle) is usually about 12 degrees.

Considering the overall beach angle α made by the line joining the crest to the step, it seems possible to divide it into two component angles: the "friction angle," α_0, which remains approximately constant at 14 degrees for all values of R up to 400 and perhaps higher, and which is independent of the absolute size of the beach particles; and a "porosity increment," α_p, which appears to depend only on the absolute size of the material. The porosity increment varies between zero for fine sand and about 8 degrees for shingle.

(4) That the porosity increment depends only on the absolute size of the grains, and not on the wave-dimension, seems reasonable from the following known facts. The velocity of percolation through material of grain diameter d, where d is so small that the flow can be considered to be viscous, is given by:

$$v = \frac{Ag}{\nu} \cdot \frac{H}{L} d^2,$$

where H denotes the head of water, L the length of the path, and ν the kinematic viscosity of the fluid, which can be taken as 0·01 for water. Since the beach angle remains of the same order for a very large range of wave-amplitudes, the length L is approximately proportional to the crest height s, which is itself proportional to the wave-amplitude. Furthermore, the mean value of the head H is also nearly proportional to the wave-amplitude. Hence $\frac{H}{L}$ is approximately constant, and so for fine material with low porosity:

$$v \propto d^2.$$

For coarse material with high porosity (pebbles), on the other hand, the corresponding expression is:

$$v = B\sqrt{g \cdot \frac{H}{L} d},$$

whence:

$$v \propto \sqrt{d}$$

The porosity therefore increases very rapidly as the material changes fro fine sand to pebbles, but varies little throughout the range of beach-shingl sizes.

That the friction angle α_0 is independent of the wave-amplitude can b verified experimentally. The plane beach of Fig. 12, Plate 1 (fine sand was thrown up by waves of 11 centimetres amplitude ($R = 220$). Subse quently the surface was disturbed by a train of short choppy waves o only 1·5 centimetre amplitude ($R = 30$). Erosion took place, and th removed material collected farther down to form a step, as shown in *Fig. 17* The surface of the excavation was, however, still plane, and it ran exactl parallel to the 14-degree plane of the original beach. (It is worth notin that, since R was less than 45, no ripples were formed on the lower beach even though the material was fine sand.)

(5) The lateral instability of the beach crest when the surge top reache the junction between a porous beach and an impervious layer (wall o plate) is easily explained by the percolation hypothesis. If the free

Fig. 17.

surface surge reaches the exposed layer before all of it has fallen throug the intervening pebbles, a surface backwash is created where there was n backwash before. This backwash removes the uppermost pebbles an so lowers the crest. On the next surge the backwash is greater still because the impervious layer is exposed and water has surged over i without any loss by percolation. More pebbles are removed, and so on until the crest is submerged to such an extent that percolation through i is negligible, that is to say, the beach angle becomes that of an imperviou beach—14 degrees.

Similarly, if by chance a group of pebbles lodges temporarily at th crest, it causes, locally, a greater separation between the top of the fre surge and the impervious layer. Consequently there is increased percola tion through the group of pebbles, the return surge is reduced in intensity and so more pebbles are added to the existing group. The beach therefor builds up at this point.

(6) The effect of porosity on the behaviour of a shingle beach may hav interesting applications to the problem of protecting shorelines from se erosion. For instance, if it is required to induce the maximum possibl accumulation of shingle, the presence of an impervious wall appears to b

undesirable. It would seem preferable to allow the shingle to pile up against an open palisade of vertical steel bars placed parallel to, and in front of, the sea wall which is to be protected.

The Effect of Lateral Water-Movement over Sand Beaches.

This Paper has, so far, dealt with experimental beaches formed by surges which move in two dimensions only, that is to say, with those whose lateral velocity at right-angles to the direction of advance of the wave is negligible. In the case of large shingle of high porosity, observation on the seashore shows that when the direction of advance is perpendicular to the shore-line the waves do indeed retain this same direction throughout the rise and fall of the surge. Experimental shingle beaches thrown up within the narrow confines of the wave tank appear, for this reason, to imitate very closely the profiles of real shingle beaches in nature.

When, however, attempts are made to imitate in the tank the formation of sand beaches, the correspondence between the artificial and the natural beaches is not so close. The 14-degree beach thrown up in the tank (Fig. 12, Plate 1) is considerably steeper than most sand beaches found in nature. The reason for this discrepancy is suggested by two interesting observations, one (*a*), in the wave tank, and the other (*b*), in nature.

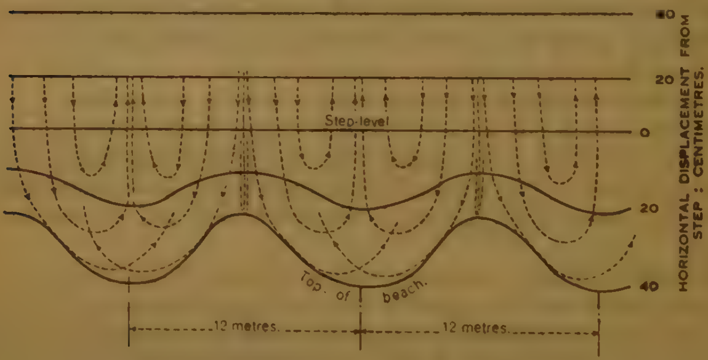
(*a*) Fig. 18, Plate 1, shows an apparently stable beach formed in 0·05 centimetre sand from an originally flat surface standing at 4 degrees by a series of waves of 6 centimetres amplitude which had so long a wave-length that they did not break. From the outset an instability was noticed. For though the beach contours were repeatedly straightened out by hand during the wave action, a low bank was, before long, always formed on one side or the other; this bank caused the surge to assume a circular sweeping motion across the tank instead of moving directly up and down as it had done on other beaches. Such a bank can be seen at the top of the beach in the figure. Measurement will show that its surface is inclined at the 14-degree angle. It lies on the camera side of the tank, and the lateral movement of the surge was away from it and across to, and then down, the other side of the tank, where there was no bank at all, and where the surface retained the general 5-degree inclination right up to the surge height.

(*b*) *Fig. 19* shows the ground plan of a typical sand beach near Mersa Matruh on the Egyptian coast west of Alexandria. Here an outer reef limits the wave-amplitude to about 60 centimetres, and the whole beach formation is simplified by the lack of tide in the Mediterranean and by a steady shoreward wind which blows at right-angles to the beach-line for long periods. It will be seen that the contour lines of the upper beach run in a regular succession of bays, which have an average width of about

12 metres, and are separated by promontories. The beach angle of thes promontories approaches, but never exceeds, 14 degrees. The beach angl in the middle of each bay is considerably less, and may be as low a 3 degrees. The profile everywhere is a straight line, and all these straigh lines run down to join a common horizontal and unindented step-line Below the step the lower beach bears transverse ripples as in Fig. 12 Plate 1.

Waves with their straight crest-lines parallel to the general shore-lin approach and break when they are immediately over the step. The surge which piles up thickly against the steep face of each promontory, is divide by it into two diverging streams. The upper fringe of each stream, as i flows inwards towards the hollow of a bay, surrounds and heads off tha

Fig. 19.



part of the surge which has flowed directly up the hollow. The two sid streams from the promontories on either side meet in the centre an together form a return surge down the hollow of considerably greate intensity than the previous upward surge there. It should be noted als that owing to the introduction of a sideways flow there is no longer an dead period (except on the front of each promontory) when the water an the sand grains it carries come momentarily to rest. The grains are kep in continual movement and therefore do not get a chance to settle o the bed.

There can be little doubt that both of these effects, the incipien beginnings of which were previously noted in the wave-tank experiment (Fig. 18, Plate 1), are responsible for the fact that the angle of a natura sand beach is less than the experimental 14-degree friction angle, α_0.

On beaches exposed to less constant winds, waves, and water-levels the rhythmic production of bays and promontories is not, of course, s pronounced as it is under the ideal conditions on the North African coast

The Size-Grading of the Particles over a Pebble Beach.

When using shingle of mixed grain-size—0·5 to 0·9 centimetre—a very distinct sorting-out of the stones took place during the formation of a model-beach. From the lowest point D (Fig. 3, Plate 1) at which bed movement took place, the stones remaining on the surface of the mature shelf dwindled rapidly in size towards the step, until pebbles of the minimum diameter were found by themselves as a transverse belt immediately at the foot of the steep slip-face leading up to the step. The surface material composing this face was, on the contrary, of maximum diameter. From here to half-way up the upper beach the diameter fell to the general mean diameter of the whole mass. Above this again, the diameter increased with increasing steepness of the beach, till at the crest only stones of maximum size were found.

It was noticed, however, that when a fresh layer of new upper beach was thrown up from below and deposited, the selective grading was more apparent than real, being confined, except at the very top of the crest, to the surface only. The reason for this was clear: far more material was always set in motion forwards and backwards than was deposited, and this oscillating layer was a considerable number of grain diameters in thickness. During the motion the oscillating layer itself became graded vertically, because the small grains tended to fall to the bottom of the layer, leaving only the larger ones at the top. Whenever, therefore, a small permanent deposition took place from the layer, it tended to consist only of the small grains; the largest stones remained and accumulated on the surface of the oscillating layer as more material of the general beach composition arrived from below.

Small quantities of fine material originally mixed with the pebbles disappeared from the beach altogether. All the fine grains were washed downwards and outwards by the circulation of water through the body of the beach, and accumulated on the tank floor at the foot of the shingle beach.

On the whole, it may be said that the model-beach of shingle imitates, in a remarkable way, all the features of its full-scale counterpart.

The Effect of Variations in the Still-Water Level (tides) and of Wave-Amplitude, during Wave Action.

The effect of a slowly falling tide on an existing model-beach of large shingle ($R = 30$ to 70) has been described on pp. 33, 34 under the heading "The Initial Beach Profile." The material was left standing at an angle of 19 degrees whatever the wave-amplitude might be, and the beach surface was nearly plane. It was unfortunate that time did not permit experiments to be made with fine sand at higher values of R.

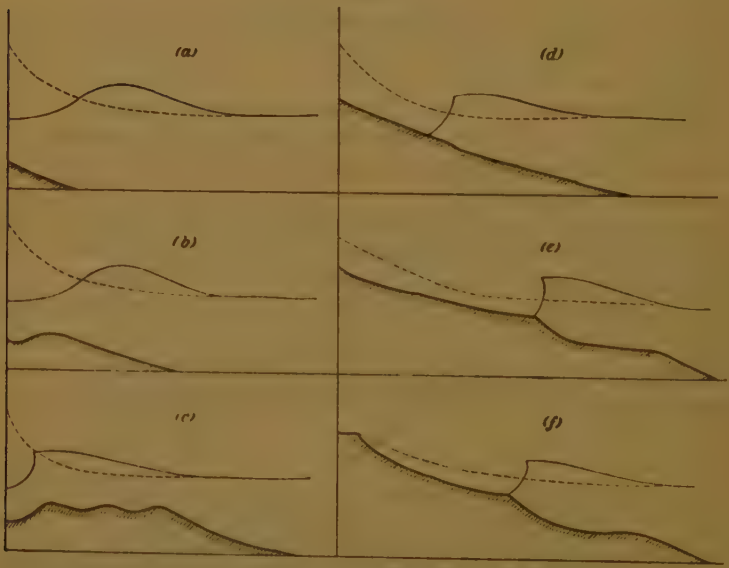
In the case of a steadily-rising tide, it is clear that at any given momen the beach has not reached the mature profile corresponding to the wav and water conditions at that moment, but that the profile approache maturity as the rate of rise is made slower in relation to the frequency o the wave action. In this connexion it may be noted that the experiment brought out that the rate of bulk movement of beach material by wave of a given amplitude varies with the diameter of the grains. Each wav moves a far greater weight of coarse material than of fine material.

Since, for any given material, the beach form depends on the wave amplitude, and since it does not approach maturity till many waves hav acted on it, the profile of a natural beach which is formed by a successio of waves whose amplitude varies at random cannot be as clear cut as th profile of a model-beach formed in the wave tank.

Submerged Beaches at the Foot of a Vertical Wall.

All the foregoing refers to beaches of which the quantity of materia was initially large. Experiments were made to see what happened t

Figs. 20.



the beach profile when a small embryo beach at the foot of a wall was fe with a continuous supply of material, and was allowed to grow progress ively bigger. The results showed that the growth of such a beach i discontinuous, and passes through at least two critical stages.

Figs. 20 show the successive stages which were noticed. Materia

(0·7 centimetre) was fed in gradually at the foot of the beach, and was worked up it by the continued action of the waves. (The wave-amplitude remained the same throughout the experiment, but although in stages (*c*) to (*d*) (*Figs. 20*) the position of the break did not alter appreciably, there was no break at all in stages (*a*) and (*b*) because of the feeble distortion of the wave caused by the little embryo beach. In these early stages the wave merely surged vertically up the wall as a true *clapotis*.)

It will be seen that at stage (*b*) the beach had risen to such a height that the scour caused by the collapse of the *clapotis* down the wall had formed a trench in the beach immediately against the wall. As more material was added this scouring action at the wall prevented the beach there from rising any higher. The material therefore accumulated farther out. The growth of this accumulation presently caused the wave to break before reaching the wall. At this stage (*Fig. 20* (*c*)) a chain of two or three acceleration ripples appeared (Fig. 21, Plate 1). These travelled slowly outwards from the scour trench, which was consequently deepened and widened.

This ripple system appeared to be unstable, for it broke down without the addition of any more material. When the scour trench had grown to a maximum size, the accumulated material farther out was suddenly and rapidly moved forward to fill it up and obliterate it. The beach once more rose to a crest against the wall; but now the crest was exposed at low-water. The profile is shown in *Fig. 20* (*d*). This new crest rose steadily; but concurrently a fresh accumulation of material began farther out, so that the beach angle grew smaller. A second critical stage was reached when the beach angle attained the minimum value of 14 degrees (stage (*e*)). This stage appears to correspond with the condition described on p. 40 and in Fig. 15, Plate 1. Finally the addition of more material caused another sudden forward movement of the beach. The beach angle steepened to 21–22 degrees, and the shingle crest, now piled up to the maximum height of the surge, formed a porous barrier between the surge and the wall. After this final stage (*f*) the addition of more material merely had the effect, already described, of making the whole beach advance to seaward without change of profile. Fig. 22, Plate 1, shows a typical submerged beach formed with the fine 0·05-centimetre material.

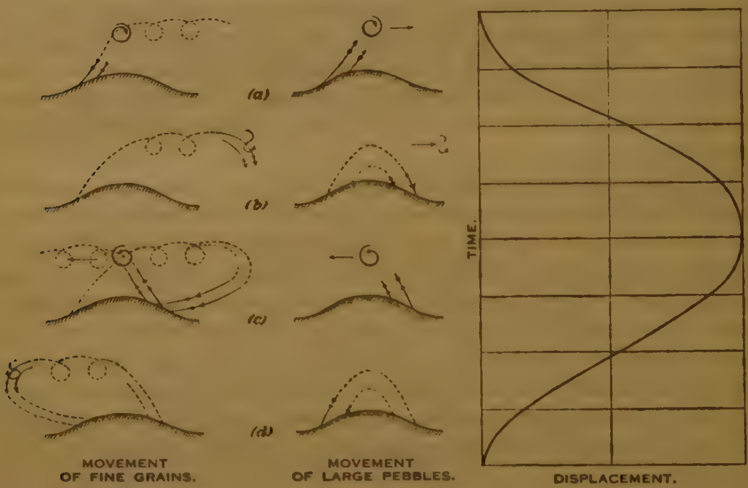
Acceleration, or Oscillation Ripples.

The formation of long continuous regular ripples on the surface of a nearly horizontal sheet of beach sand is familiar to everyone. But the great similarity in form between these beach ripples and the familiar wind ripples seen on the surface of a sand dune has led to the commonly-held view that the same mechanism is responsible for both. This is most

unlikely, for two reasons : (*a*) the wind ripple is formed under a steady uni-directional flow of the air, whereas the conditions under which the beach ripple is formed are invariably those of an oscillatory bed movement caused by the passage of wavelets through the water above ; (*b*) ripples formed on the bed of a steadily-flowing stream of water, although they may assume a variety of shapes, never resemble either the wind or the beach ripple.

Transverse beach ripples have been produced in the wave tank not only with sand, but also with shingle up to 0·9 centimetre in diameter.

Figs. 23.



With this large material the movement, both of the individual grains an of the surrounding water, can be watched with great ease.

The peculiar motion set up during each stroke of the water oscillatio results without doubt from the acceleration of the water mass rather tha from its acquired velocity of flow. The motion is utterly unlike that see during sand-ripple formation under steady-flow conditions either in water-flume or in a wind-tunnel.

Successive phases of the motion, in the cases of both fine and larg grains, during one complete wave cycle are sketched on the left-hand sid of *Figs.* 23. The displacement–time curve on the right indicates the phas of the cycle to which the sketch on the left refers.

Phase (*a*). The initial acceleration causes a violent surge up the lef hand side of each ripple. This removes grains from the surface an carries them high into the fluid above. At the same time the fluid abov acquires a rapid clockwise spin about a horizontal axis parallel to th ripple direction. The rising grains are swept into the vortex. If they a

small they are carried round and round inside it; larger grains appear merely to move round through the upper semicircle before dropping out again. In both cases the mean linear velocity of the water carries the vortex across to the opposite side of the ripple.

Phase (*b*). Retardation of the general linear water-motion has the effect of checking the angular velocity of the vortex. The grains leave it and begin to fall. By the time the linear velocity has ceased at the end of the stroke the vortex has begun to turn inside out and to disintegrate. The falling grains may or may not have reached the ripple surface, depending on their rate of fall. In the case of fine grains they may be carried far beyond the precincts of their own ripple.

Phase (*c*). If the grains are large they have already fallen on to the right-hand side of the same ripple from which they rose. If they are small they are still falling. The reverse motion of the water carries them back, so that they still fall on to their own ripple. In the meantime the acceleration of the water and the consequent surge up the right-hand side of the ripple has picked up other grains and carried them high into the fluid above. The vortex, now anti-clockwise in direction, passes over the ripple and involves the rising grains in its motion.

Phase (*d*). Retardation destroys the vortex; grains fall towards the surface. Large grains reach the left-hand side of their own ripple before the end of the stroke; fine grains are carried on to the left beyond their own ripple, but return again during the following phase (*a*).

The formation of these acceleration ripples displays several interesting features: (*a*) If the water oscillation is symmetrical, the grains tend to remain associated with their own particular ripple. In this case the ripples remain stationary. If the oscillation is not symmetrical there is a drift of grains from ripple to ripple, but it is not yet clear whether this must always involve a movement of the ripple itself. (*b*) The mechanism allows of great constancy of ripple wave-length over a very wide range of grain size and oscillation amplitude. When the grain size is small the grains are carried farther, but their low terminal velocity allows of their being carried back again to their own ripple before they reach the surface. If the period of oscillation remains of the same order, an increase in wave-amplitude involves an increase of acceleration at the beginning of each stroke. Furthermore, since the effect of acceleration appears to be to raise the surface grains vertically upwards, an increase of amplitude therefore causes them to rise higher than before. The water-displacement carries them farther during phases (*a*) and (*b*), but they remain longer in the fluid and have more time to be carried back to their own ripple during phase (*c*). (*c*) The alternate creation and destruction of large and violent vortexes must absorb and dissipate a considerable amount of energy from the layers of water near the bed surface. This probably accounts for the rather surprising fact that the level of the shelf does not fall when the size of the particle is reduced. There seems to be some automatic adjustment

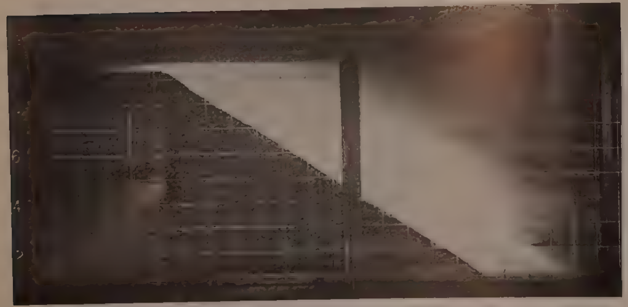
whereby the damping effect of the ripples on the bottom water-movement in each case just counteracts the increasing mobility of the smaller grains.

The Paper is accompanied by eight sheets of drawings and by eighteen photographs, from which Plate 1 and the Figures in the text have been prepared.

Fig. 1.

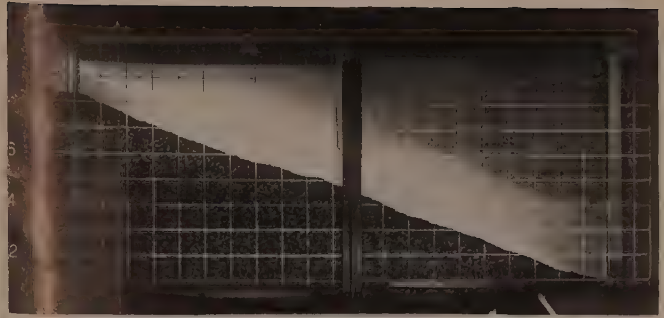
EXPERIMENTAL WAVE-TANK.

Fig. 3.



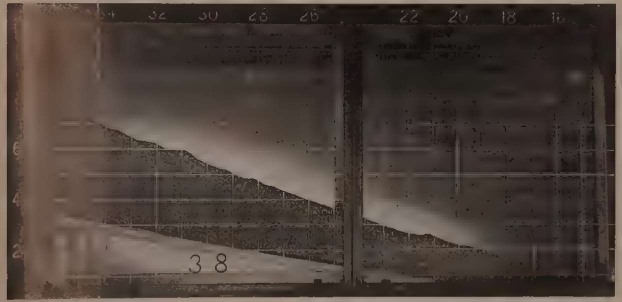
THE ANGLE OF REPOSE OF A BEACH OF 0·7-CM. MATERIAL.

Fig. 4.



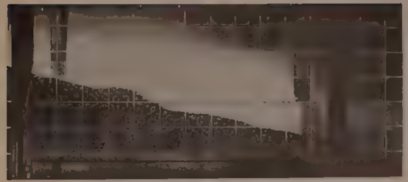
THE RESULT OF WAVES (FALLING TIDE) ON A BEACH OF 0·7-CM. MATERIAL (BEACH ANGLE 19°).

Fig. 5.



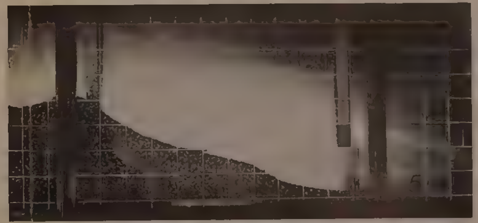
THE RESULT OF WAVES (FALLING TIDE) ON A BEACH OF 0·3-CM. MATERIAL (BEACH ANGLE 19°).

Figs. 6.



Wave amplitude: $h = 4$ cm.

Wave amplitude: $h = 11{\cdot}3$ cm.



Wave amplitude: $h = 18$ cm.

BEACHES FORMED OF 0·7-CM. MATERIAL. STILL-WATER HEIGHT 30 CM.

Figs. 7.

Wave amplitude: $h = 4{\cdot}1$ cm.

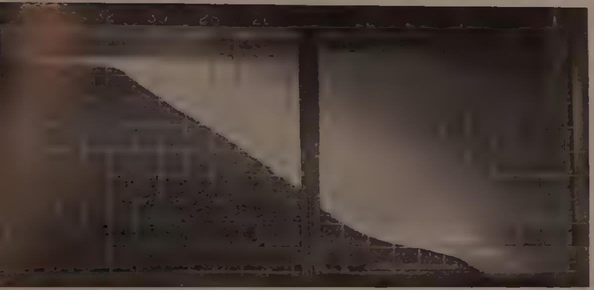
Wave amplitude: $h = 16{\cdot}2$ cm.

Wave amplitude: $h = 23$ cm.

BEACHES FORMED OF 0·7-CM. MATERIAL. STILL-WATER HEIGHT 50 C

William Clowes & Sons, Limited: London.

Fig. 8.



BREAKDOWN BEACH: $H_0 = 20$ cm., $h = 14$ cm.

Fig. 10.

A FLAT SHELF IN 0·7-CM. MATERIAL. NO RIPPLES. WAVE AMPLITUDE: $h = 29$ CM.

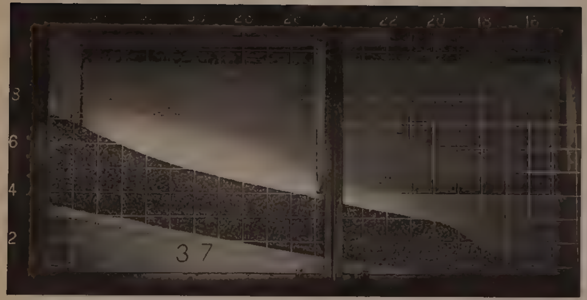
Fig. 11.

THE FORMATION OF ONE RIPPLE ON THE SHELF. GRAIN SIZE 0·3 CM. WAVE AMPLITUDE: $h = 27$ CM.

Fig. 12.

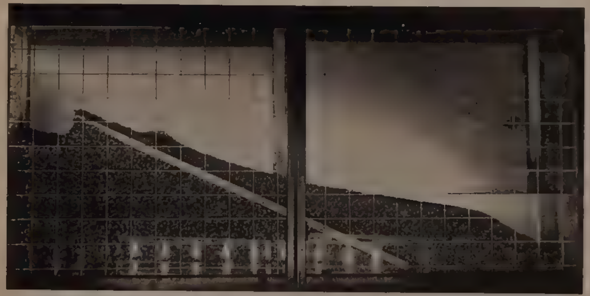
A BEACH OF 0·05-CM. MATERIAL. $H_0 = 20$ cm., $h = 11$ cm., $\alpha = 14°$.

Fig. 15.



BREAKDOWN OF THE BEACH WHEN SURGE REACHES THE WALL. THE BEACH ANGLE FALLS TO 14°.

Fig. 16.



THE EFFECT OF AN IMPERVIOUS LAYER UNDER THE BEACH CREST. INSTABILITY AND BREAKDOWN.

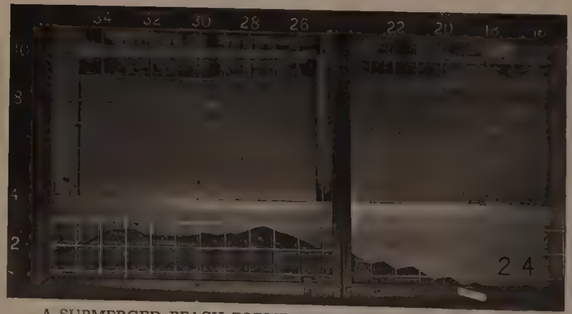
Fig. 18.

A MODEL SAND BEACH SHOWING THE LOW BEACH ANGLE (5°) DUE TO LATERAL WATER MOVEMENT.

Fig. 21.

RIPPLED STAGE IN THE FORMATION OF A SUBMERGED BEACH OF 0·7-CM. MATERIAL.

Fig. 22.



A SUBMERGED BEACH FORMED WITH 0·05-CM. MATERIAL.

R. A. BAGNOLD.

"The Prevention of Coast Erosion."

By Francis Maurice Gustavus Du-Plat-Taylor, M. Inst. C.E

Introduction.

In the British Isles erosion is continuous in some parts of the coasts, notably in the east, whilst accretion is taking place in other parts, such as the north-west. It was stated before the Royal Commission on Coast Erosion in 1911 that, over a period of 25 years, the loss had been 31,000 acres and the gain 30,752 acres. Owing to preventive measures it is certain that the rate of loss has been much diminished since that date; but the fact remains that the loss is usually of good agricultural land, and sometimes buildings, whereas the gain is only of sand and shingle.

Large artificial gains have been made by reclamation, such as that of the Zuyder Zee abroad, and at Dublin, Southampton, and elsewhere in the British Isles; but these notes are not concerned with this class of work.

Measures to stop erosion include sea-walls (whether merely earthen embankments or structures of timber, stone or concrete), groynes, or planting.

The Author does not propose to deal with planting, which is a very wide subject and as to which it is usual to consult botanical specialists in any particular case.

Sea-Walls.

In agricultural districts an earth-bank is the usual form of sea-wall, and hundreds of thousands of acres in the Thames estuary, on the Wash, and elsewhere, are thus protected.

The standard Thames sea-walling is of clay, or earth faced with puddled clay, on which block chalk is pitched 6 or 8 inches thick. It has a seaward slope of 1 in 2 and a rearward slope of 1 in 1, and the width at the top is usually 6 feet. The crown is kept up to the statutory levels of 18 to 20 feet above Ordnance Datum. Much of the walling dates from Roman times.

In Holland, a much flatter slope is adopted for the embankments, varying on the seaward side from 1 in 3 to 1 in 12. The banks are formed of sand, protected by basalt pitching up to 15 inches thick, pinned down by short piles or stakes. The landward slopes are usually covered with stone, stake, and wattle-work or straw-thatching.

Another form of protection is the use of timber breastworks, consisting of sheet-piles driven in the beach, combined with groynes in front.

These methods are not much less costly, and are much less durable, than concrete walls; and they spoil the amenities of the beach if it is used by the public.

In Holland, labyrinths of projecting timber stakes are frequently driven at about mean tide-level, either in the beach or in the seaward slopes of pitched embankments. These are intended to break up the waves, and they are very effective. Many of these labyrinths are to be seen in the island of Walcheren, where they are inoffensive as regards access to the sea, since the population there is sparse.

Where, however, urban areas have to be protected, it is not usual to rely on earthen embankments or on timber-works, and some form of stone or concrete wall is generally adopted. Such sea-walls were formerly made with a vertical or slightly battered face, and many examples may be seen at seaside towns. Such sea-walls can be successful only where the regime of the beach is maintained by groyning, as the undertow resulting from the wave-stroke against a vertical wall in an on-shore gale will speedily remove the sand or shingle in front of the toe of the wall to a great depth, and if this is not restored by the action of groyning, a collapse may ensue.

Walls of this type have been frequently breached, and only those of considerable thickness and with very deep foundations can be considered really safe. The design of such a wall is not comparable to the requirements of a dock or quay-wall. In one instance, to the Author's knowledge, the level of the beach in front of a sea-wall was reduced by 11 feet in one season by a series of on-shore gales.

Furthermore, in the case of vertical-faced walls the waves are thrown up to a great height in gales, and many tons of water descending on the promenades behind them tend to break up the paving or surfacing, so that the material behind such walls may be washed out.

Modern sea-walls are now usually constructed with sloping, or preferably stepped, faces, so that the force of the waves is gradually absorbed in travelling up the stepped slope, and the water is finally turned back towards the sea by a parapet-wall with a curved face. The road or promenade is usually situated immediately behind this parapet-wall.

Where space is restricted, this sloping or stepped wall may be surmounted by an open arcade carrying the promenade or road. A recent example of this type of construction will be described later.

Stepped-type walls are usually constructed of reinforced concrete of moderate thickness, with a heavy toe in front, and, to avoid any disturbance of material beneath them, it is always necessary to carry down a curtain-wall, or line of steel sheet-piling, from the toe into some impervious stratum.

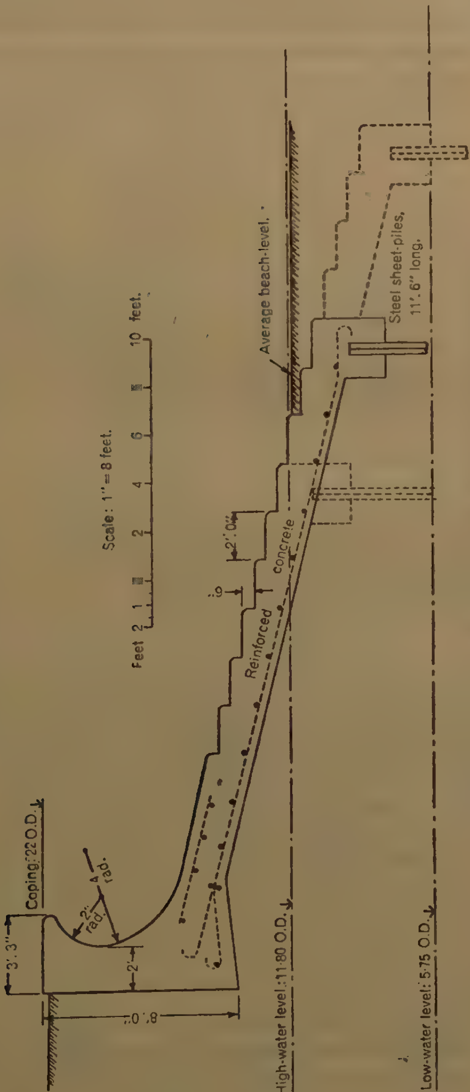
Land-water behind the parapet-wall must be dealt with either by a system of drains or by weep-holes in the wall. One advantage of the stepped type of wall is that, in the event of the level of the beach being

Figs. 1.

STEPPED TYPE OF SEA-WALL AT NEW ROMNEY, KENT.

lowered by exceptional gales at some time subsequent to the construction of the wall, the step-work can be extended to the new level; whereas a vertical wall would have to be underpinned or rebuilt.

Fig. 2.



Cross-Section of Sea-Wall at West Bay, Portrush.

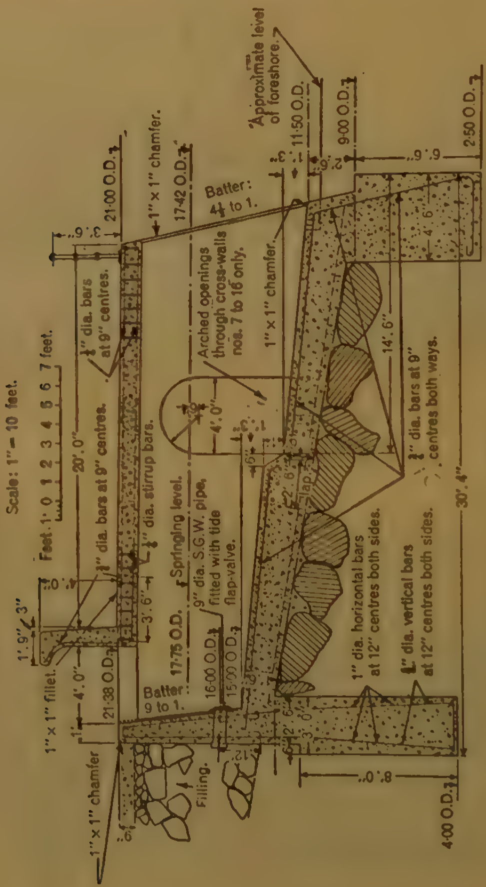
Figs. 1 illustrate a wall of this type, constructed to the Author's designs at New Romney in Kent. The wall is of various widths to accord with variations in the inclination and level of the beach, the total length

being about 2 miles. A continuous line of steel sheet-piling, of various lengths, and of copper-content steel, was driven under the toe of the wall into the clay underlying the beach material. Case-type low groynes were installed along the frontage.

Fig. 2, p. 55, is a cross-section of another wall of this type, designed for preventing erosion and protecting a proposed promenade and road at the

Fig. 3.

Scale: 1" = 10 feet.



Cross-Section of Sea-Wall at Folkestone.

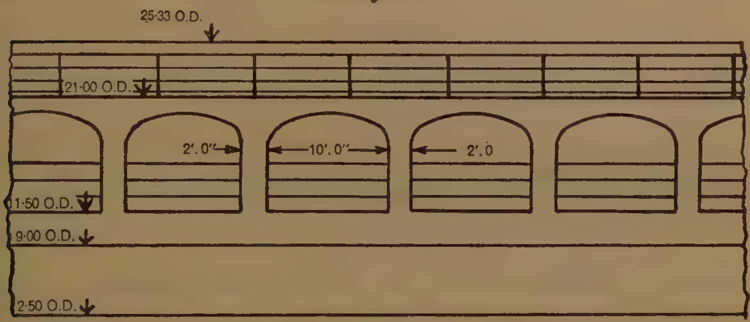
West Bay at Portrush, Northern Ireland. The length of the proposed wall is 3,300 feet. The conditions there are that, though the bay is exposed to the full force of Atlantic gales, the sand beach is very flat and the low-water mark is a long way out from the line of the wall. The waves are largely absorbed in traversing this wide stretch of sand; hence it has been possible to adopt a light section for the wall.

The width of the wall and the length of the piles under the toe vary

at different parts of the frontage, the greatest width of wall being in the centre of the bay.

The erosion at Portrush is of two kinds, sea erosion and wind erosion, both of which attack the sand dunes and slopes at the top of the beach, endangering buildings and roads behind. The wall is intended to arrest sea erosion, and the wind erosion will cease when the area of sand behind the wall is covered with roads, gardens, and house property. The scheme also includes several adjustable screw groynes. Owing to the war, the inception of the work has been postponed indefinitely.

Fig. 4.



Scale: 1" = 16 feet.

Step-type walls afford easy access to the beach for bathers and others, and this is of importance at seaside places whose prosperity depends largely on the amenities provided.

An example of the arcade type of wall, recently completed at Folkestone in front of the East Cliff, is illustrated in *Figs. 3* and *4*.

Here, a promenade 20 feet wide is carried on arches surmounting a sloping and stepped wall. The latter is provided with continuous curtain-walls at back and front, going down to a solid foundation in the Folkestone Beds. At the back of the promenade is a screen wall, 4 feet high, behind which openings are formed in the deck to allow water thrown over the screen wall to pass back to the sea. The curtain-wall in the rear is carried up to form a retaining-wall behind the arches. 9-inch diameter weep-pipes fitted with tidal flap-valves are provided in this retaining wall in the centre of each arch. The whole structure is of reinforced concrete.

In all long sea-walls of any type it is necessary to provide contraction joints at suitable intervals. These may be straight joints with opposing V-grooves packed with a bitumen and sand composition; or copper-strip joints may be utilized. The former require attention and repacking from time to time. The latter require no attention, but they are more costly.

Groynes.

Probably more diverse opinions have been expressed on the subject of groynes than on any other matter, both as to their height and length and to their siting.

In many seaside towns groynes are built of concrete or masonry, and are of a substantial nature and proportionately costly; but, generally speaking, groynes are constructed of timber.

The old-fashioned high groyne with heavy oak spurs still persists in many places. It accumulates material to a great height, and hence there is a deep drop on the lee side, often as much as 15 feet. Thus the beach is accumulated in a series of waves normal to the foreshore line. The low type of groyne produces a much more uniform accumulation, but the groynes must be placed much closer together. In Holland, low stone groynes, built over fascine mattresses, have proved very satisfactory in dealing with the material there, which is exclusively sand. These groynes are 40 to 80 feet wide, and stand 8 to 10 feet above the beach-level. In Great Britain, low groynes are usually of the case type, consisting of planking between pairs of posts embedded in concrete. Such groynes are often only 4 feet high above the beach-level.

Planks can be added between the posts as the beach builds up. When the groyne becomes completely buried, the posts and concrete bases can be dug up and refixed in a fresh position. In one form of low groyne, shown in *Figs. 5*, screw-piles are substituted for the posts, and the groyne can be adjusted to the beach-level from time to time by screwing or unscrewing the piles by means of a special capstan.

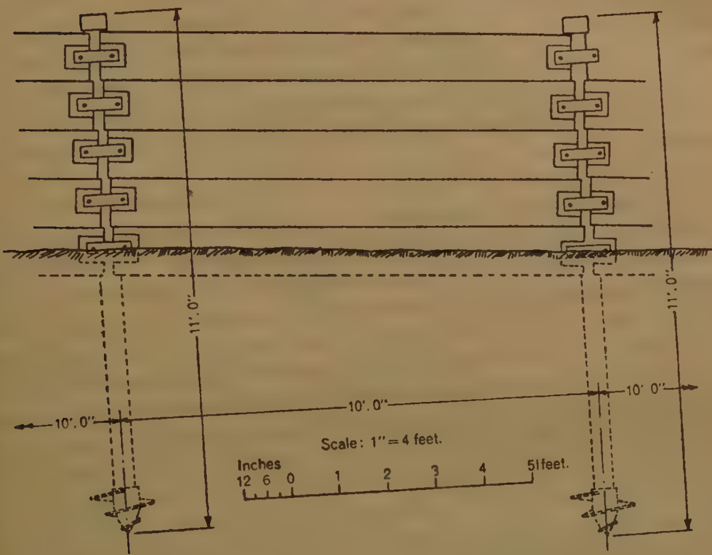
The siting of groynes always requires careful study. If time and money are available, float observations should be made to ascertain the direction of the prevailing currents at various states of the tide; but often this is not possible. Most valuable information can be obtained by consulting local coastguards, harbour-masters, and fishermen.

Generally, groynes should, in the Author's opinion, be aligned normal to the prevailing current, and their distance apart should be roughly equal to their length. There are, however, wide differences of opinion on this matter, both as to length and alignment of groynes, as will be evident from the perusal of many Papers on the subject, and handbooks on sea-defences.

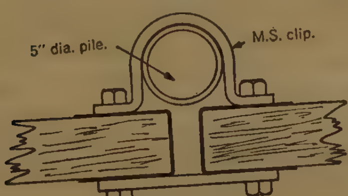
The maintenance of groynes is always a heavy item of expenditure, but it is very important, as the stability of a sea-wall may be endangered if the groynes are not kept up.

It is sometimes thought by local authorities and private owners that, once groynes are installed, no further expenditure need be incurred; whereas actually (except in the case of heavy masonry or concrete groynes) about 7½ per cent. per annum on the initial cost is required for the proper upkeep of groynes. At Wicklow, for instance, a sea-wall and system of

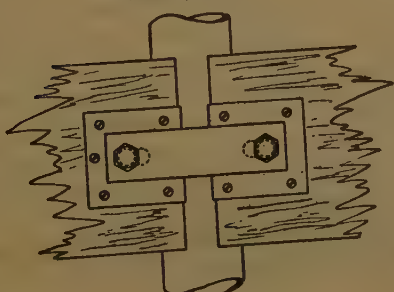
Figs. 5.



Adjustable Screw-Pile Groyne.

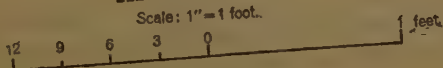


PLAN OF JOINT.



ELEVATION OF JOINT.

Scale: 1" = 1 foot.



groyning was provided in 1906 to the designs of the late Sir Alexander Rendel. No provision was made by the Government of Ireland for maintenance either of the wall or of the groynes. The latter were gradually disintegrated by gales and heavy seas, and by 1927 they had totally disappeared. The level of the beach having consequently fallen below that of the foundation of the sea-wall, this became completely wrecked. In consequence a good deal of land, and some houses, have been lost.

In the classic case of Jackson *versus* The Eastbourne Local Board (1884), the plaintiff was contractor for the construction of a sea-wall for the defendants, but not for the construction of groynes. There were some existing groynes, and the plaintiff relied upon the verbal assurance of the defendants' surveyor that these would be maintained and that others would be made. In fact, the groynes were not maintained and no new ones were made ; and the effect of this neglect upon the construction of the sea-wall was to lower the beach-level by the sea removing the shingle, in consequence of which a large part of the new sea-wall fell down before the works were completed. The plaintiff sued the Local Board for the cost of maintenance and damages, but lost his case in the House of Lords, on the ground that there was no warranty as to the groynes in the contract.

No contractor in our day would undertake work of this sort unless he was indemnified against the risk of the foundation of the wall being removed by the sea. The action of the sea is capricious and uncertain, and the fact that a beach has maintained its level for 20, 30 or even 50 years is no guarantee that it may not unexpectedly be depleted at any time. Even with suitable and efficient groynes, a series of on-shore gales may remove much beach material into deep water, and, as stated above, a beach was in one instance lowered 11 feet in one season, though in the two succeeding seasons the original level was restored.

The Author would emphasize that this class of work cannot be compared with dock and harbour work in still waters, and provision must always be made against unexpected conditions arising. Unfortunately there is generally only a limited amount of money available for sea-defence works, which are not in themselves profit-earning. The engineer has therefore so to design the works as to provide the maximum degree of security for the money available, and it is impossible for him to guarantee the permanent security of the works, particularly in cases where he is not made responsible for their maintenance.

The adjustable screw-pile groyne illustrated in *Figs. 4* was constructed by Messrs. Braithwaite & Co., Engineers, Ltd.

This communication is accompanied by four sheets of drawings and by two photographs, from some of which the Figures in the text and the half-tone page-plate have been prepared.

LATE CORRESPONDENCE
ON PAPER PUBLISHED IN
JUNE 1940 JOURNAL.

Paper No. 5230.

"Highway Transition-Curves: A New Basis for Design."[1]

By Hugh Alan Warren, M.Sc. (Eng.), Assoc. M. Inst. C.E.

Mr. J. M. Fenton observed that about 7 years ago he inserted a number of sinusoidal transition-curves when realigning curves on a section of the main line of the East Indian railway. That transition-curve was extremely close in form and characteristics to that of the Author and, for all practical purposes, might be considered as identical with it. That curve was designed, as was the Author's, to eliminate the sudden acquisition of a rate of change of curvature at the commencement of the transition, and an equally sudden loss at its junction with the circular arc. Although that transition gave perfect running, Mr. Fenton had discontinued its use and had returned to the spiral, chiefly because the ganger was reluctant to allow the superelevation to be acquired in any but a uniform manner.

On railways, vertical accelerations were much less than on highways as, with the inner rail following the grade-line, the total extra-grade vertical movement of the outer rail was limited to about 6 inches, whereas on a highway, even when the road was tilted about its centre, a vehicle on the outer side might suffer a lift of several feet over the transition. The relative importance of vertical accelerations on highway transitions was therefore much greater than in the case of railway transitions, and a form of curve to minimize the forces to which they gave rise was highly desirable.

The method employed by the Author to arrive at the formula for his curve was ingenious; but after stipulating the need for the curved shape of the longitudinal profile shown in *Figs. 1* (*c*) and (*d*) (p. 374§), it appeared strange that the harmonic form which they would have to assume, or at least approximate to, was not used by the Author as the basis of his analysis. In the sinusoidal transition, the analysis of which followed, a

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[1] Journal Inst. C.E., vol. 14 (1939–40), p. 373 (June 1940).

§ Page numbers so marked refer to the Paper (Footnote 1, above.).—Sec. Inst. C.E.

harmonic relationship between the length of transition and the curvature (or superelevation) was assumed.

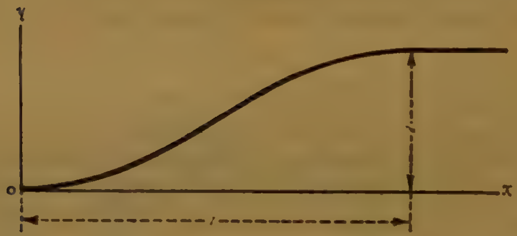
Fig. 3 represented the curve relating the length and curvature of a sinusoidal transition of length L and ultimate curvature D. Its equation was :—

$$y = \frac{D}{2}\left(1 - \cos\frac{\pi x}{L}\right),$$

and the rate of change of curvature was

$$\frac{dy}{dx} = \frac{D}{2} \cdot \frac{\pi}{L} \sin\frac{\pi x}{L}.$$

Fig. 3.



That was a maximum when $\sin\frac{\pi x}{L} = 1$,

that was to say, when $x = \frac{L}{2}$

and the value of $\frac{dy}{dx} = \frac{\pi}{2} \cdot \frac{D}{L}$.

Thus the maximum rate of change of curvature occurred at the mid-point of the transition, and was $\frac{\pi}{2} = 1{\cdot}57$ times the constant rate of change of curvature of a spiral of equal length and ultimate curvature.

Denoting the equation of the transition by $y = f(x)$,

when $\frac{dy}{dx}$ was very small, $\frac{d^2y}{dx^2} = \frac{1}{R}$;

then $$\frac{d^2y}{dx^2} = \frac{D}{2}\left(1 - \cos\frac{\pi x}{L}\right) = \frac{1}{2R}\left(1 - \cos\frac{\pi x}{L}\right) \quad . \quad . \quad . \quad (1)$$

Integrating, $$\frac{dy}{dx} = \frac{1}{2R}\left(x - \frac{L}{\pi}\sin\frac{\pi x}{L}\right) + C;$$

but $$\frac{dy}{dx} = 0 \text{ when } x = 0, \quad \therefore C = 0$$

$$\therefore \frac{dy}{dx} = \frac{1}{2R}\left(x - \frac{L}{\pi}\sin\frac{\pi x}{L}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

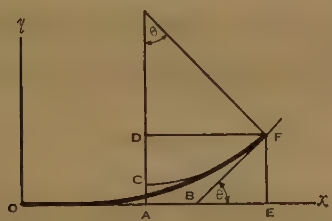
Integrating, $$y = \frac{1}{2R}\left(\frac{x^2}{2} + \frac{L^2}{\pi^2}\cos\frac{\pi x}{L}\right) + C_1;$$

but $$y = 0 \text{ when } x = 0, \quad \therefore C_1 = -\frac{L^2}{2\pi^2 R},$$

and the equation for the transition was

$$y = \frac{1}{2R}\left(\frac{x^2}{2} + \frac{L^2}{\pi^2}\cos\frac{\pi x}{L}\right) - \frac{L^2}{2\pi^2 R} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Fig. 4.



When $$x = L,$$

$$y = \frac{1}{2R}\left(\frac{L^2}{2} + \frac{L^2}{\pi^2}\cos\pi\right) - \frac{L^2}{2\pi^2 R}$$

$$= \frac{L^2}{R}\left(\frac{1}{4} - \frac{1}{\pi^2}\right)$$

$$= 0{\cdot}1487\,\frac{L^2}{R}.$$

That was very nearly nine-tenths the value of $\frac{1}{6} \cdot \frac{L^2}{R}$ for a spiral of equal length and ultimate curvature.

To obtain the shift.

From equation (2), when $x = L$,

$$\frac{dy}{dx} = \frac{1}{2R}\left(L - \frac{L}{\pi}\sin\pi\right) = \frac{L}{2R}.$$

Referring to *Fig. 4*,

$$\frac{dy}{dx} = \tan\theta = \frac{L}{2R}$$

when θ was small, $\tan\theta = \theta = \sin\theta$;

$$\therefore\ \mathrm{DF} = R\sin\theta = R\,.\,\frac{L}{2R} = \frac{L}{2};$$

also $$\mathrm{CD} = \frac{(\mathrm{DF})^2}{2R} = \frac{L^2}{8R};$$

$\therefore$ shift
$$= \mathrm{AC} = \mathrm{AD} - \mathrm{CD} = \mathrm{EF} - \mathrm{CD}$$
$$= \frac{L^2}{R}\left(0{\cdot}1487 - \frac{1}{8}\right)$$
$$= 0{\cdot}0237\frac{L^2}{R}.$$

That was very nearly six-tenths the value of $\frac{1}{24}\cdot\frac{L^2}{R}$ for a spiral of equal length and ultimate curvature. That value of the shift was almost identical with $\frac{1}{40}\cdot\frac{L^2}{R}$, the value deduced by the Author for his curve.

From equation (3), the value of y, when $x = \frac{L}{2}$,

$$= \mathrm{AB} = \frac{1}{2R}\left(\frac{L^2}{8} + \frac{L^2}{\pi^2}\cos\frac{\pi}{2}\right) - \frac{L^2}{2\pi^2 R}$$
$$= \frac{L^2}{R}\left(\frac{1}{16} - \frac{1}{2\pi^2}\right)$$
$$= 0{\cdot}0118\frac{L^2}{R};$$

namely, half the value of the shift.

The close agreement of the sinusoidal transition with that of the Autho was demonstrated in the Table below, wherein comparison was made between the ordinates of the two curves for the first example with which the Author had illustrated his Paper. For further comparison the ordinates of a spiral were also given.

x	0	20	40	60	80	100	120	130
Sinusoidal curve	0·000	0·002	0·030	0·147	0·440	1·003	1·913	2·513
Author's curve .	0·000	0·002	0·033	0·157	0·460	1·030	1·947	2·551
Spiral curve . .	0·000	0·010	0·082	0·277	0·656	1·282	2·215	2·817

A comparison between the vertical accelerations and the rate of gai of radial accelerations would further illustrate the close relation betwee the two forms of transition.

The vertical acceleration was

$$\frac{mwv^4}{g} \cdot \frac{d^4y}{dx^4} = K\frac{d^4y}{dx^4}.$$

For the Author's curve that was

$$K\left\{\frac{6}{RL^3}(L - 2x)\right\},$$

and its maximum value, when $x = 0$ or L,

was $$\pm 6 \cdot \frac{K}{RL^2}.$$

For the sinusoidal transition the vertical acceleration was

$$K\left(\frac{1}{2R} \cdot \frac{\pi^2}{L^2} \cos \frac{\pi x}{L}\right),$$

and its maximum value, when $x = 0$ or L,

was $$\pm \frac{\pi^2}{2} \cdot \frac{K}{RL^2}.$$

The two maxima given above bore to each other the ratio of

$$6 : \frac{\pi^2}{2}, \text{ or } 1 : 0{\cdot}82.$$

Thus, for transitions of equal length and ultimate curvature, the maximum vertical acceleration of the sinusoidal transition was about four-fifths that of the Author.

The rate of gain of radial acceleration was

$$v^3 \cdot \frac{d^3y}{dx^3}.$$

For the Author's curve that was

$$v^3 \cdot \frac{6x}{RL^3}(L - x),$$

and its maximum value, when $x = \frac{L}{2}$, was $3 \cdot \frac{v^3}{2RL}$.

For the sinusoidal transition the rate of gain of radial acceleration was

$$v^3 \cdot \frac{\pi}{2RL} \sin \frac{\pi x}{L},$$

and its maximum value, when $x = \frac{L}{2}$, was $\pi \cdot \frac{v^3}{2RL}$.

The relation between those two values was

$$3 : \pi, \text{ or } 1 : 1{\cdot}05.$$

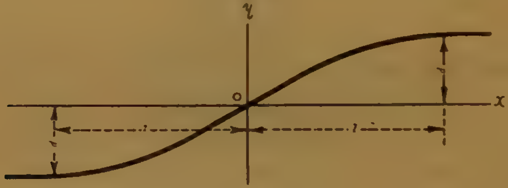
5

In the relatively unimportant consideration of the rate of gain of radial acceleration, there was, therefore, little to choose between the two.

The sinusoidal transition-curve, for the same limiting value of the vertical acceleration, was shorter than the transition-curve proposed by the Author, and was therefore preferable.

Neither of those two curves was entirely suitable for use in the case of continuous and reversed curves without any intervening straight. In such cases the longitudinal profile should take the form shown in *Fig. 5.*

Fig. 5.



The equation of the transition-curve resulting from that profile was

$$y = \frac{2L}{\pi R}\left(x - \frac{2L}{\pi} \cdot \sin \frac{\pi x}{2L}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

and the maximum vertical acceleration was

$$\frac{\pi^2}{4} \cdot \frac{K}{RL^2},$$

or half the value for the sinusoidal transition.

The maximum rate of gain of radial acceleration was

$$\pi \cdot \frac{v^3}{2RL},$$

or the same as for the sinusoidal transition.

Unlike the spiral and sinusoidal transitions, the shift did not occur at the mid-point, but was distant $\left(1 - \frac{2}{\pi}\right)L$, or $0{\cdot}363L$, from the commencement, its value being $0{\cdot}0277\frac{L^2}{R}$.

For the example cited above, the ordinates of that transition-curve were :

x	0	20	40	60	80	100	120	130
y . .	0·000	0·016	0·127	0·424	0·984	1·872	3·132	3·909

The analyses given by Mr. Fenton were particular solutions of the general problem of reversed curves of different radii. The general profile

was illustrated in *Fig. 6*, wherein L denoted the combined length of the two transitions, L_1 and L_2, between curves of curvature $D_1 = \frac{1}{R_1}$ and $D_2 = \frac{1}{R_2}$.

The equation to the transitions, based upon co-ordinates OX, OY, was

$$y = \left(\frac{1}{R_1} - \frac{1}{R_2}\right)\frac{x^2}{4} + \left(\frac{1}{R_1} + \frac{1}{R_2}\right)\frac{L^2}{2\pi^2}\cos\left(\frac{\pi x}{L} + \cos^{-1}\left[\frac{R_2 - R_1}{R_1 + R_2}\right]\right)$$
$$+ \frac{Lx}{\pi\sqrt{R_1 R_2}} - \left(\frac{1}{R_1} - \frac{1}{R_2}\right)\frac{L^2}{2\pi^2} \quad . \quad . \quad (5)$$

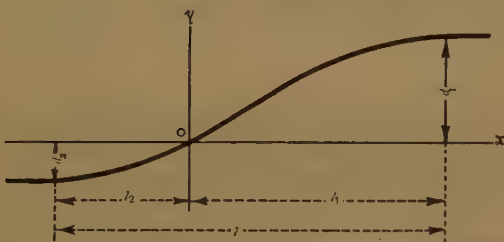
The transition lengths were

$$L_1 = \frac{L}{\pi}\cos^{-1}\left[\frac{R_1 - R_2}{R_1 + R_2}\right]$$

and

$$L_2 = \frac{L}{\pi}\cos^{-1}\left[\frac{R_2 - R_1}{R_1 + R_2}\right].$$

Fig. 6.



The shifts occurred at the following respective distances from *O* :

$$L\left\{\left(\frac{1}{\pi}\cos^{-1}\left[\frac{R_1 - R_2}{R_1 + R_2}\right]\right)\left(\frac{1}{2} + \frac{R_1}{2R_2}\right) - \frac{1}{\pi}\sqrt{\frac{R_1}{R_2}}\right\}$$

and

$$L\left\{\left(\frac{1}{\pi}\cos^{-1}\left[\frac{R_2 - R_1}{R_1 + R_2}\right]\right)\left(\frac{1}{2} + \frac{R_2}{2R_1}\right) - \frac{1}{\pi}\sqrt{\frac{R_2}{R_1}}\right\}.$$

The general expressions for the shifts were rather long, but their values could be readily derived for any particular example in the graphical manner previously adopted.

For the particular case of a transition between a straight line and a curve, that was, where $R_2 = \infty$ and $R_1 = R$, equation (5) reduced to the form

$$y = \frac{1}{2R}\left(\frac{x^2}{2} + \frac{L^2}{\pi^2}\cos\frac{\pi x}{L}\right) - \frac{L^2}{2\pi^2 R},$$

which was identical with equation (3) previously obtained.

Again, in the special case of transitions between reversed curves of

equal radii, that was, where $R_1 = R_2 = R$, the general equation was simplified to

$$y = \frac{L}{\pi R}\left(x - \frac{L}{\pi}\sin\frac{\pi x}{L}\right),$$

which was identical with equation (4) when $2L$ was substituted for L.

The following numerical example illustrated the application of the general equations :—

$R_1 = 1{,}000$ feet ; $R_2 = 2{,}000$ feet ; $L = 213 \cdot 757$ feet ; $L_1 = 130$ feet ; $L_2 = 83 \cdot 757$ feet.

x	y	
−83·757	−0·838	Transition to 2,000-foot radius curve.
−60	−0·331	Transition-length 83·757 feet.
−40	−0·103	Shift occurred 29·41 feet from commencement, and its value was
−20	−0·013	0·010 foot.
0	0·000	
20	0·014	
40	0·115	Transition to 1,000-foot radius curve.
60	0·388	Transition-length 130 feet.
80	0·916	Shift occurred 49·39 feet from commencement, and its value was
100	1·764	0·488 foot.
120	2·983	
130	3·738	

The values selected for that example had been specially chosen so that the transition to the 1,000-foot radius curve was 130 feet long, and comparison was therefore possible with former illustrations.

The general equations were of universal application for all cases of simple and reversed curves, as well as where, in short curves, there was no intervening circular arc. The maximum vertical accelerations were less than in any other acceptable form of transition-curve.

The Author, in reply, stated that Mr. Fenton's observations were interesting because he had used vertical acceleration as a basis for design, as well as, or perhaps in place of, the usual criterion of rate of gain of radial acceleration. The curve advocated by Mr. Fenton was very similar to that proposed by the Author, as might be expected, since it was based upon almost identical premises. The Author had considered the sinusoidal expression for the longitudinal profile, but had discarded it in favour of the method used. As between the two curves obtained, the relative advantages were so slight as to be negligible ; indeed the exact type of curve used was of little moment, the important point being that the real basis of design was vertical acceleration, not the conception hitherto used and known as C. Many engineers had so shaken themselves free of the traditional value for C of 1 foot per second per second per second, as to use 2, or even to speak with daring of 3, whilst a few now realized that for highway curves the conception of C was simply not a basis of design at all.

since it could with impunity assume values of up to 50 or more, whereupon other considerations, such as vertical acceleration, became predominant. The Author was much more concerned with the principle that vertical acceleration should be used as a basis of design, than with the exact shape of the curve, especially when it was borne in mind that for highway curves, unlike railway curves, there was no guarantee that a motorist would follow, or even try to follow, any predetermined curb-line. With regard to the figures given by Mr. Fenton on p. 64, *ante*, making a comparison between the ordinates for a sinusoidal, the Author's, and a spiral transition, it was evident that a motorist would indeed need to be skilled, both as a mathematician and as a driver, deliberately to follow the curve of his choice.

The Author had preferred his expression for the ordinate y to that developed by Mr. Fenton from a sinusoidal profile, simply because it contained only powers of x, instead of the cosine function involved in Mr. Fenton's expression. The " cubic parabola " had long been preferred to the spiral or lemniscate, for a very similar reason.

Mr. Fenton's analyses regarding reverse and compound curves were both interesting and useful ; similar expressions could easily be developed for the Author's type of curve.

OBITUARY.

SIR THOMAS HUDSON BEARE, youngest son of Mr. T. H. Beare, of Netley, South Australia, was born on the 30th June, 1859, and died at his home in Edinburgh on the 10th June, 1940. He was educated at Prince Albert College and University, Adelaide. He won the South Australian Scholarship in 1880 and proceeded to England, to continue his studies at University College, London. In 1887 he was appointed Professor of Engineering at the Heriott-Watt College, Edinburgh, and in 1889 he filled the vacant Chair of Engineering at University College, London. In 1901 he returned to Edinburgh as Regius Professor of Engineering to the University; in 1908 he was elected to the University Court of Edinburgh, becoming Dean of the Faculty of Science in 1913. He remained in the University until his death. From 1897 until 1902, and again from 1909 until 1913 he was honorary secretary to the Institution Committee on Tabulating the Results of Steam-Engine and Boiler Trials, and compiled the reports which were printed in volumes 150 and 195 of the Minutes of Proceedings.

He was elected an Associate Member of The Institution in May, 1885, and was transferred to the class of Member in February, 1893.

He presented a Paper on "The Building-Stones of Great Britain: their Crushing Strength and other Properties"[1], for which he was awarded a Telford premium. He was president of the engineering section of the British Association in 1922; served for several terms as vice-president of the Royal Society of Edinburgh; and was also an honorary life member of the Institution of Mechanical Engineers and a Vice-President of the Institution of Structural Enginers.

In addition to his professional activities, Sir Thomas was a Justice of the Peace and Deputy Lieutenant of the City of Edinburgh. He was Chairman of the Edinburgh University Officers' Training Corps Committee, and since 1908 had been one of the two representatives of Edinburgh University on the city's Territorial and Air Force Association. He also represented Scotland on the Miners' Welfare Commission.

In 1885 he married Louise, daughter of Alexander Newman, who survives him.

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[1] Minutes of Proceedings Inst. C.E., vol. 107 (1891–2, Part 1), p. 341.

WILLIAM VAUX GRAHAM was born on the 23rd April, 1859, and died on the 22nd May, 1940, in Westminster Hospital. He was a pupil of, and later assistant to, Mr. R. W. Peregrine-Birch, M. Inst. C.E. His activities included many important sewerage schemes, including surveys of the Thames estuary in connexion with the Royal Commission on Metropolitan Sewage Discharge, 1884–5. His principal work was in connexion with public water-supplies. He was an authority on problems of underground water, and was often retained to give evidence before Parliamentary Committees in regard to the effect of pumping upon existing wells, etc. In 1918 he was one of the engineers selected by the Board of Trade to report upon the water-power resources of Great Britain. He was consulting engineer to a number of water authorities, and was responsible for the construction of works in many parts of England. He also acted as resident engineer and later as engineer for the water-power works for the British Aluminium Company, at Foyers; and in collaboration with the late Mr. J. C. Hawkshaw, Past-President Inst. C.E., was responsible for the Vigeland water-power works in Norway.

Mr. Graham was elected a Member of The Institution in May, 1896. He was also a Fellow of the Royal Meteorological Society.

In 1916 he married Ethel Stow, daughter of James Young, of Cadboll, Fearn, Ross-shire, who survives him.

WILLIAM HENRY HAMER was born on the 5th September, 1869, at Darwen, Lancs., and died at Bridlington on the 14th May, 1940. He was educated at Rugby, and served his pupilage under Mr. E. G. Mawbey, M. Inst. C.E. From 1889 to 1894 he was engaged as an assistant to the chief engineer of the Hull docks and railways, and also carried out surveys and soundings for the Humber Conservancy. In 1895 he was appointed resident engineer at Tilbury docks, and also acted as engineer at Tilbury for the East and West India Company sea-wall. In 1898 he became resident engineer at the Victoria and Albert Docks, London. In 1903 he was appointed engineer-in-chief to the Harbour Board of Auckland, N.Z., and designed the new scheme of harbour and railway works adopted by the Board. He retained that position until his retirement in 1925. He was also consultant for several Australian harbours. During the Great War he served on a mission to the United States and Canada in connexion with the supply of coal and oil in bulk to Australasia.

Mr. Hamer was elected an Associate Member of the Institution on the 3rd December, 1895, and was transferred to the class of Member on the 28th March, 1905.

PROFESSOR CHARLES FREWEN JENKIN, C.B.E., F.R.S., was born at Claygate, Surrey, on the 24th September, 1865, and died at St. Albans, Herts., on the 23rd August, 1940. He was educated at Edinburgh Academy and University and at Cambridge University. After an apprenticeship on the London & North Western Railway at Crewe, he was appointed assistant to the works manager. Later he became mechanical assistant superintendent at the R.G.P.F., Waltham Abbey; resident engineer at Nettlefold's Steel Works, Newport, Mon.; and assistant works manager to Messrs. Siemens Brothers, with whom he stayed for 10 years, rising to works manager and head of the railway department at Stafford. In 1908 he was appointed to the newly-formed chair of Engineering Science at Oxford University, and it was largely by his tact and skill that the difficulties of the first few years were overcome and the erection of the school buildings—completed in 1914—was sanctioned by Congregation. In 1929 he resigned to pursue researches into vibrational fatigue. During the Great War he was head of the branch responsible for aircraft materials of all kinds, with the rank of Lieutenant-Colonel; and with the object of preserving the mass of experimental data accumulated, he prepared the "Report on the Materials of Construction in Aircraft and Aircraft Engines", which was published by H.M. Stationery Office in 1920. Another important publication was "Earth Pressure Tables" (H.M. Stationery Office, 1934). In 1919 he was awarded the C.B.E.

As a Student of The Institution Jenkin was awarded a Miller Scholarship. He was elected an Associate Member in February, 1891, and was transferred to the class of Member in January 1912. He presented several Papers to The Institution, for which he was awarded a Telford gold medal, a Telford premium, and a Watt medal.

In 1889 he married Mary, youngest daughter of the late Lord Mackenzie, and had two sons and one daughter. The sons predeceased him.

WALTER SÉRY NICHOLSON was born in London on the 24th November, 1855, and died at his home at Surbiton, Surrey, on the 27th May, 1940. In 1877, soon after completion of his pupilage, he joined Messrs. T. and C. Hawksley as an assistant, and on the death of Mr. K. P. Hawksley, M. Inst. C.E., in 1924, he was taken into partnership. He remained with the firm until his death—a period of nearly 63 years, during which he was responsible for a large number of waterworks carried out by them, including the design and construction of large reservoirs for the Southend Water Works Company, the Durham County Water Board, the Bristol Water Works Company, and the South Essex Water Works Company. He also advised numerous water-supply authorities on problems of additional resources and the construction of the necessary work, superintended the

preparation of schemes for Parliamentary deposit, and gave expert evidence before Select Committees of both Houses of Parliament.

Mr. Nicholson was elected an Associate Member of The Institution in 1881, and was transferred to the class of Member in 1908.

In 1881 he married Eliza M. Brettell, daughter of Walter Brettell, who survives him, and had one son.

ROGER THOMAS SMITH was born at Forest Hill, London, on the 23rd March, 1863, and died in London on the 28th April, 1940. He was educated at Mill Hill School and at University College, London, and served his pupilage in mechanical engineering with Messrs. Hathorn, Davey & Company, of Leeds. In 1890 he went to India, where he took charge of the erection of pumping machinery in several large cities, and later became municipal and waterworks engineer at Allahabad. On his return to England, he was appointed resident engineer at the Davies Street generating station. After service in Belgium as technical manager to the *Compagnie Hydro-electrique Anversoise*, he joined the firm of Sir Alexander Kennedy and Jenkin, by whom he was employed on the electrification of tramways for the London County Council and for the city of Buenos Aires, and also on the electrification of the Hammersmith and City Railway. In 1905 he was appointed electrical engineer to the Great Western Railway, and occupied that position for nearly 20 years, during which period he was responsible for numerous installations of electrical machinery for harbours, stations, bridges, and goods yards; for the installation and maintenance of electric lighting on rolling stock, and for the design and electrical track equipment of the Ealing and Shepherd's Bush Railway. In 1924 he retired and became a partner in the firm of Highfield and Roger Smith, consulting engineers.

He was elected to Associate Membership of The Institution in April, 1889, and was transferred to the class of Member in April, 1918. In 1911 he presented a Paper on "Electric Lighting of Railway Trains: the Brake Vehicle Method"[1], for which he was awarded a George Stephenson gold medal. He was also a Past-President of the Institution of Electrical Engineers, of the Institute of Transport, and of the British Section of the *Société des Ingénieurs Civils de France*, and a member of the Institution of Mechanical Engineers, of the American Institute of Electrical Engineers, and of the Senate of the University of London.

In 1902 he married Margaret Isabel, eldest daughter of the late Dr. G. Carey Foster, F.R.S., who survives him.

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[1] Minutes of Proceedings Inst. C.E., vol. clxxxvii (1911–12, part 1.), p. 142.

SIR JOSEPH JOHN THOMSON, Kt., O.M., F.R.S., was born a Cheetham Hill, near Manchester, on the 18th December, 1856, and die at Cambridge on the 30th August, 1940. He was educated at Owen College, Manchester. He entered Trinity College, Cambridge, in 1876 and his connexion with that College remained unbroken until his death In 1880 he was second wrangler and Smith's prizeman and was elected t a Fellowship. He was appointed a Lecturer in Mathematics in 1883 and in 1884 succeeded the late Lord Rayleigh as Cavendish Professor o Experimental Physics in the University. In 1918 he was appointed b the Crown as Master of Trinity College. His work in the Cavendis Laboratory brought him world-wide fame and he received honorar degrees from numerous British and foreign universities, with the honorar Fellowship or membership of many learned societies. He was awarde the Royal, Copley, and Hughes medals of the Royal Society ; the Hodgkin medal of the Smithsonian Institute ; the Franklin medal and Scott medal Philadelphia ; the Mascart medal, Paris ; the Dalton medal, Manchester the Faraday medal of the Institution of Electrical Engineers ; and th Kelvin medal. In 1906 he was awarded the Nobel prize for physics.

It was not only as a physicist that he was remarkable, but also a the inspirer and teacher of brilliant pupils. Under his direction th Cavendish Laboratory became a source for the supply of physicists t fill professional posts all over the world.

He was the author of numerous publications on physics, electricity and magnetism, and to him belongs by general consent the chief credi for the discovery of the electron, which in its turn was responsible fo many of the advances in physical science.

He was knighted in 1908, and was awarded the Order of Merit i 1912. The honour of burial in Westminster Abbey was accorded, an the ceremony was attended by representatives of every branch of science

Sir Joseph Thomson was elected an Honorary Member of The Institu-tion on the 2nd December, 1924. He was President of the British Associa-tion in 1909 ; and from 1916 to 1920 was President of the Royal Society.

He married, in 1890, Rose Elisabeth, daughter of the late Sir Georg Paget, K.C.B., and had one son and one daughter.

ABSTRACTS OF THE CURRENT TECHNICAL LITERATURE OF ENGINEERING AND APPLIED SCIENCE.

ENGINEERING CONSTRUCTION.

A Study of the Allen Salt-Velocity Method of Water Measurement. M. A. Mason (*J. Boston Soc. Civ. Engrs.*, 27, *207–235; July 1940*).—The method, devised by Professor C. M. Allen, was described by him in 1923 (Engng. Abstracts, Part 19, No. 9; April 1924). The Author now describes and presents the results of a study made at the University of Grenoble, France, to define the field of application of the Allen method and the possibility of its use in all types of conduits or channels. He first develops a physical conception of the method and then endeavours to prove this conception by experiment. Tests were made in two adjoining concrete channels. The injection apparatus was similar to that usually employed, and the design permitted modification or substitution of the various injection valves. The results are presented in the form of curves, oscillograms, and Tables, and in his conclusions the Author discusses the principal factors to be considered in the use of the method for precise measurements.

Model Studies of Overflow Spillway Sections. A. S. Offitzeroff (*Civ. Engng., N.Y.*, 10, *523–526; Aug. 1940*).—The Author describes two series of model-tests on overflow spillways of modified Creager section, varying the upstream and downstream face-angles. The first series of experiments, on unsubmerged spillways, was made to determine m, the coefficient of discharge in the expression $Q = mL\sqrt{2g}\left(H + \frac{V^2}{2g}\right)$ as a function of α_1 and α_2, the angles formed by the upstream and downstream faces respectively with the horizontal. Curves showing the relationship established between m, α, and H, and a Table relating m to α_1 and α_2 for the design value of H, are reproduced. The second series of tests was made on submerged spillways. m_n, the ratio of discharge under submerged conditions to discharge under unsubmerged conditions, was plotted against $f\left(\frac{D}{H}\right)$, where D denotes the elevation of the downstream water-level above

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Notes.—An asterisk prefixed to a reference, thus *Civ. Engng. N.Y.*, denotes that the article is illustrated.

The abbreviated titles of periodicals are those used in the "World List of Scientific Periodicals" (Oxford 1934).

the crest, and H denotes the head for unsubmerged conditions. The Author concludes that the function $m_n = f\left(\frac{D}{H}\right)$ is influenced by the shape of the spillway, and that the previous experiments by the U.S. Deep Water Board underrate the capacity of a submerged spillway over the range of $\frac{D}{H}$ from 0 to 0·75, and in most cases overrate the capacity for values of $\frac{D}{H}$ between 0·75 and 1·0.

Glass Strip as a Reinforcing Medium for Concrete. (**Civ. Engng.*, *Lond.*, 35, *239–240 ; Aug. 1940.*)—The results of tests in which glass was substituted for steel as a reinforcement for concrete are presented. They show that slabs of glass-reinforced concrete would carry four times the maximum load required by the Home Office for street and other air-raid shelters. The glass reinforcement is in strips $\frac{5}{16}$ inch thick, the depth being half that of the beam which it is to reinforce. One edge of the glass is not cut, but is the fire-finished edge, known as the selvedge, in the state in which it comes from the process. It is concluded that glass provides a good substitute for steel ; but it should not be used in cases where there is likelihood of impact loading.

Reinforced Portland Cement Concrete in Cold Weather. R. V. ALLIN and H. H. TURNER (**Concrete Constr. Engng.*, 35, *434–438 ; Sept. 1940*).—The Authors' experiments were made with the object of deriving approximate working rules for pre-heating, applicable to the English climate. They were of two types : (*a*) cubes were wholly or partially frozen and subsequently crushed after various periods of maturing ; (*b*) cubes were made at various temperatures by preheating the water only, and were exposed to conditions likely to occur when concreting in frosty weather. The crushing strength of the cubes was then determined at various ages. Field tests were also made on concrete deposited in cold weather followed by frost at night. The results are plotted in curves. They confirmed the fact that concrete frozen before setting will harden to a large extent if undisturbed, but does incur permanent loss of strength, which may attain 50 per cent., depending upon the degree of freezing and to a smaller extent upon the water/cement ratio. The Authors conclude that the life of reinforced concrete depends chiefly upon the preservation of its cover, which is the part most vulnerable to attack by frost.

Construction Plant for Friant Dam, California. (**Engng. News-Record*, 125, *40–44 ; 1 Aug. 1940.*)—The Friant dam, 20 miles north of Fresno, on the San Joaquin river, is of the concrete gravity type, straight in plan, and is 3,340 feet long and 325 feet high, with a volume of 2 million cubic yards

The construction plant includes means for the economical handling of aggregate and the simultaneous discharge by seven-car trains of 65-cubic-yard bottom-dump cars. All concrete goes out at one level over a high steel trestle under hammerhead cranes and revolving derricks. Ice-machines are used for cooling the mixing water.

Deepening Bridge Foundations by Means of H-Section Steel Bearing Piles. (**Rly. Age, Chicago*, 109, *178–182 ; 3 Aug. 1940.*)—By more than doubling the length of bridge waterways, extending spans to minimize obstruction to flow, and anchoring piers and abutments on H-section steel piles driven to 70 feet or more below rail-level, the Union Pacific Railway has afforded protection to four important bridges over the Mojave river in southern California. The operations at each bridge are described in detail.

The Merritt Parkway Bridge, Connecticut. W. G. Grove (**Engng. News-Rec.*, 125, *328–331 ; 12 Sept. 1940*).—Two unusual features characterize this new bridge over the Housatonic river between Stratford and Milford, Conn. The first is the use of canopy-type towers, designated " one-leg bents ", at two of the piers. This form of structure was adopted in order to shorten the span required over a channel that is skewed with the centre-line of the bridge. The second is the use of an open-grid steel deck for the entire length of the bridge, namely, 1,824 feet between abutments. The bridge is of the plate-girder type, with spans ranging from 128 feet to 224 feet, and provides two 26-foot roadways. Details of design and construction are given.

Coded Track Circuits on the Pennsylvania Railroad. (**Rly. Signaling*, 33, *427–434 ; Aug. 1940.*)—Automatic block signalling has been installed on 110 miles of double-track main line between Bradford, Ohio, and Anoka Junction, Indiana. The most interesting feature is the use of coded track circuit controls, thus eliminating line control circuits and obviating stray current trouble. The coded track circuits are of the direct-current type, and the control system as a whole is operated from batteries. This type of circuit operates successfully on track circuits 11,000 feet in length, so that the number of track circuits is minimized. The line includes long sections of tangent with only fourteen curves, none of which exceeds 1 degree 20 minutes. The automatic blocks range in length from about 3 miles to a maximum of 4 miles. Typical normal and reverse code direct-current track circuit diagrams are reproduced.

Reducing Curve Resistance by Rail Lubrication. (**Rly. Age, Chicago*, 109, *255–256 ; 17 Aug. 1940.*)—In a report presented by a subcommittee to the American Railway Engineering Association, descriptions are given of dynamometer-car tests carried out between Utah Junction, Colorado, and the east portal of the Moffat tunnel, over 45·63 miles of

track, whereon fourteen mechanical lubricators had been installed by the Denver and Salt Lake Railway. Three test runs were made before any grease was applied to the rails. The lubricators were then charged with grease, and after 14 days, during which period about 200 trains had passed, three more test runs were made. A continuous dynamometer record was made of the drawbar-pull (train resistance) and the speed of the train during each run, and from this record tabulations were made at about 150 mile-post locations on each run. From the results it is concluded that rail and flange lubricators will effect a reduction in train resistance and cause an increase in tonnage ratings where gradients on curves control the main load.

A 48-inch Diameter Cast-iron Water-main. (**Engng. News-Rec.*, **125**, *214–218; 15 Aug. 1940.*)—The new water-supply for Wichita, Kansas, will comprise a subsoil supply of 32 million gallons per day from twenty-five wells about 30 miles north-west of the city; gathering-lines from the wells and a supply main to the city; and a 32-million gallon-per-day filtration plant. The most interesting feature is the cast-iron supply main, ranging in size from 20 inches to 48 inches. The design takes account of static pressure, water-hammer, foundry tolerance, and corrosion, and the computations include the external load imposed by earth-pressure and trucks and the effect upon such loads and the resulting stresses in the pipe barrel of different methods of pipe-laying, bedding, and tamping. The operations of trenching, pipe-laying, caulking, and tamping are described in detail.

New Water-Supply Pressure-Tunnel at Baltimore, Maryland. LEON SMALL (**Engng. News-Rec.*, **125**, *346–350; 12 Sept. 1940*).—The new tunnel, 7 miles in length, which connects the raw water supply with the filtration works, has a capacity of 275 million gallons per day. The inside finished diameter is 12 feet. It is lined with concrete for two-thirds of its length, and about 12,000 feet is lined with continuous welded steel plate grouted inside a previously-deposited concrete lining. The design is discussed, and details are given of the arrangements for unwatering the tunnel, including a 1,250-horse-power motor, operating a two-stage centrifugal pump with a capacity of 10 million gallons per day when delivering against a net head of 570 feet.

Novel Design Features of the Lansing Water-Conditioning Plant. C. R. ERICKSON (**J. Amer. Waterw. Ass.*, 32, *1259–1309; Aug. 1940*).—The city of Lansing, Mich., obtains its water from seventy wells in sandstone and shale. The water is hard, scale-forming, and often red owing to its corrosive properties, whilst it contains considerable iron and bacteria. A water-conditioning plant was therefore constructed and placed in operation in December 1939. This is really two units in parallel, each hal

having a maximum rated capacity of 15 million gallons per day. Equalization or interchange gates between the first carbonization basin effluents, the second settling basin effluents, and the filter effluents, together with control and separate metering of the common raw water supply give additional flexibility to the plant either for conditioning treatment or for periodical maintenance cleaning. The various operations of air and water conditioning are described in detail, and the results obtained are presented in Tables.

MECHANICAL ENGINEERING.

Some Aspects of the Design of Large High-Pressure Steel Valves. F. D. Cotterman and R. E. Falls (**J. Amer. Soc. Nav. Engrs.*, 52, *353–368 ; Aug. 1940*).—The Authors observe that the accurate determination of the proper metal thickness and the working stresses in valve bodies and bonnets is a difficult problem ; more especially when valves for high-temperature service are concerned, because of the presence of creep and the changes which take place in the properties of the materials. They describe a method of supplementary approximate calculations by tests in order to determine the thickness of metal for safe pressure-temperature rating. They also describe tests and give comparative results for the resistance to steam-flow in valves of various types.

New Turbo-Generator for the Walnut Power-Station, Ohio. C. A. Butler, jun. (**Elect. World, N.Y.*, 114, *70–73 ; 83 ; 397–399 ; 457–458 ; 13 July and 10 Aug. 1940*).—The equipment of the Walnut power-station comprised turbo-generator capacity of 72,000 kilowatts, sixteen 444-horse-power boilers, and four 1,035-horse-power boilers ; the operating conditions were 235 lb. per square inch gauge pressure and approximately 580° F. steam-temperature in the main headers. The new unit, which combines topping and condensing operations, is a tandem-compound turbine connected to a hydrogen-cooled generator. It will operate for part of the time on the regenerative cycle with steam conditions of 1,250 lb. per square inch, 28·5-inch vacuum, with feedwater heated in four stages to 395° F. On this cycle 37,500 kilowatts at unity power-factor can be developed. For the rest of the time the machine will be a superposition unit, with 49 per cent. of the steam extracted at 250 lb. per square inch for use in the low-pressure part of the plant, 27 per cent. extracted at four points for heating the combined feed-water, and only 25 per cent. passing to the condenser. The installation involved the solution of special problems in the supply of steam and circulating water.

The Mulajore Power-Station, Calcutta. (**Elect. Times*, 98, *143–144 ; 151 ; 29 Aug. 1940.*)—A detailed description is given of the new 60,000-kilowatt station, situated on the river Hooghly. The boiler-house

contains eight Babcock-Wilcox horizontal water-tube boilers, each o 11,430 square feet heating-surface, designed for a normal evaporation o 100,000 lb. per hour, the steam conditions being 365 lb. per square incl at 750° F. The superheater is of the double-loop interdeck type witl five rows of tubes giving a total heating-surface of 3,290 square feet. Th mechanical stokers provide a grate-area of 385 square feet. Forced an induced draught fans are included. The generating plant comprises tw 30,000-kilowatt Parsons turbo-alternators. The turbines are of the two cylinder type, taking steam at 350 lb. per square inch at 700° F. Th alternators have a maximum continuous rating of 35,300 kilovolt-amperes 0·8 power-factor, 50 cycles, with a speed of 3,000 revolutions per minute The machines generate directly at 33,000 volts. Details are given of th circulating water intake, of the switchgear, and of ancillary equipment.

Harmonic Theory of Noise in Induction Motors. W. J. Morrili (**Elect. Engng., N.Y.*, 59, *474–480 ; Aug. 1940*).—The Author presents theory of noise generation which is particularly adapted to use with smal motors. This is based upon the assumption of an air-gap having uniforn length, constant permeance, and windings distributed in the usual way i finite numbers of slots. The Author states that the theory has beer used for some time in noise investigations, and has been found to bear ou test results on a large number of motors.

Flameproof Electric Motors in Industry. (**Elect. Times*, 98, *94–96 ; 8 Aug. 1940.*)—A summary is given of the principal official publication concerning the use of flameproof electrical apparatus in industry. Hom Office memoranda, schedules of hazardous dusts, and details of explosiv gases and liquids are set out in three Tables. The gases classified are (1) methane or firedamp ; (2) pentane or petrol vapour ; (3) coal-gas o coke-oven gas (60 per cent. hydrogen ; 40 per cent. methane). Difficultie of flameproof motor design are discussed, and wiring problems ar considered.

Power-Factor Correction. (**Elect. Review*, 127, *125–126 ; 16 Aug 1940.*)—The desirability of power-factor correction is emphasized, especiall in view of problems raised by war conditions, such as metal shortage capital expenditure restrictions, delivery difficulties, and the necessit for economy in every direction. The influence on tariffs is discussed, an the choice of equipment and the best point of correction are considered.

A Criterion for Knock in Petrol Engines. R. C. Plumb and A. C. G Egerton (**Proc. Instn. Mech. Engrs.*, 143, *247–260 ; Sept. 1940*).—Th Authors review theories of the mechanism of knock and suggest a rationa form for this criterion. They describe engine tests of fuels made at incipien and heavier knock under various engine conditions, and reproduce indicato

diagrams obtained by an oscillograph indicator. They derive a semi-empirical criterion of knock which applies reasonably well under all running conditions, including change of compression-ratio, and discuss the application of this criterion to fuel rating. Ignition experiments indicated that, for weak mixtures of the two blends of paraffin hydrocarbons used, the same function is a criterion of auto-ignition. These results tend to show a similarity between the mechanism of knock in an engine and of auto-ignition in a bomb; but further tests are required to establish the connexion with greater precision.

Articulated Goods Locomotives for the Delaware and Hudson Railway. (**Rly. Age, Chicago*, 109, *207–213; 218; 10 Aug. 1940.*)—Twenty single-expansion articulated goods locomotives of the 4–6–6–4-type are being delivered for high-speed heavy service. These have cylinders $20\frac{1}{2}$ inches in diameter by 32 inches stroke, and 69-inch driving-wheels. The boiler-pressure is 285 lb. per square inch, producing a calculated tractive force of 94,400 lb.; but it is anticipated that devices installed for reducing rolling friction will cause a higher proportion of this than usual to be available for drawbar-pull. The weight on the driving-wheels is 406,500 lb., and the total weight of the locomotive is 597,000 lb., whilst that of the tender is 310,200 lb. The boiler is of the straight top type, $94\frac{11}{16}$ inches in internal diameter at the first course and 102 inches outside diameter at the largest course. The heating-surface of the flues is 1,892 square feet; of the tubes 2,864 square feet; and of the superheater pipes 1,681 square feet. The capacity of the tender is 22,500 (U.S.) gallons of water and 26 tons of coal.

Locomotive Development and Design on the South African Railways. W. A. J. Day (**J. S. Afr. Instn. Engrs.*, 39, *26–47; Sept. 1940*).—South African locomotives are larger and more powerful than any in Great Britain or in many European countries, although the gauge is only 3 feet 6 inches. Most of the line is single-track, and the official maximum speed is 55 miles per hour. Owing to severe curvature, great care has to be exercised in design; all specifications require engines to pass a test curve of 275 feet. The Author discusses bogie control, the development of the boiler, front end design, superheating, stokers, feed-water heating, valve gear, lubrication and bearings, counterbalancing, compounding and streamlining. He also deals with the three types of articulated locomotives in use on the South African railway—Garratt, Mallet, and Fairlie—with electric and diesel-electric locomotives, and with the development of the tenders.

Air-Cooled Rail-Cars for the New York, Susquehanna and Western Railway. (**Rly. Mech. Engr.*, 114, *303–309; Aug. 1940.*)—Two rail-cars, each with seating capacity for 80 passengers, are driven by an oil-engine which differs from the conventional diesel engine in employing a

positively-timed electric ignition system supplied by high-tension magnets. The engine is a six-cylinder horizontal overhead-valve unit with cylinders $6\frac{1}{4}$-inch bore by $6\frac{1}{2}$-inch stroke, capable of propelling the car, fully loaded, at a speed of 60 miles per hour at 1,800 revolutions per minute. Transmission is by a direct-drive converter containing a hydraulic torque-converter, direct and free-wheeling equipment, and a driving-shaft with midship bearing and universal joints to transmit the power to the driving axle on one axle of the truck. The air-conditioning equipment, of $6\frac{1}{2}$ tons capacity, is of the electro-mechanical type using Freon. The electrical equipment is on the dual-voltage system at 12 volts for starting, head-lights, etc., and at 125 volts for air-conditioning, heating, lighting and control accessories.

Oil-Engined Cranes for Railway Service. (**Oil Engine*, 8, *98–100*; *Aug. 1940*.)—The London Midland and Scottish Railway has put into service ten oil-engined cranes, in all of which a standardized form of engine and hydraulic coupling is employed. Their capacities range from 4 tons to 10 tons. For the 10-ton cranes, four-cylinder 88-brake-horse-power engines are used, whilst the others have three-cylinder 66-brake-horse-power units. The percentages of slip at various engine-speeds, at full-load torque and half-load torque, are plotted in curves. The cranes are used for a wide variety of railway service. Their principal advantage is that fuel cost is incurred only during actual use, whereas with steam it may be necessary to maintain pressure for many hours, during which the actual working time may be measured in minutes. All of the cranes have been designed to take magnetic equipment, if required, the current for which will be generated by separate diesel-dynamo sets.

Electrically-Heated Strand-Dipping Tank for Manufacturing Wire Rope. F. J. MAHR (**Gen. Elect. Rev.*, 43, *320–321*; *Aug. 1940*).—The tank described is of welded construction, rectangular in shape, and of such capacity that one filling of tar compound per day is ample. A dipping sheave is mounted at the top, and the wire strand, passing through the tank, is immersed in the compound. The latter is heated to 150° F. before it enters the tank, and is then heated to 270° F. for the immersion process. A screw-in type of immersion heater is used, a unit of 2·5 kilowatts being placed on each side of the dipping sheave to give even heat distribution and provide low watt density of the exposed surface of the heater. This spreads the heat over a surface sufficiently large to prevent carbonization and possible damage to the compound or the heater. A standard thermostat and a magnetic switch control the units. The Author states that wire-rope compound is applied with precision and that the lubrication of the rope is superior because automatic temperature-control ensures uniform and economical application.

MINING ENGINEERING.

Deterioration of Coal in Storage Heaps. P. Fuchs (**Archiv für Wärmewirtschaft u. Kesselwesen*, 21, *39–41; Feb. 1940*).—The Author presents the results of systematic investigations carried out on large heaps of coal stacked in the form of truncated pyramids and exposed to rain, wind, and sun. Samples taken when the coal was stacked, and also on each occasion when the supply was drawn upon, were investigated analytically and calorimetrically. The course followed by the temperature in the coal-heap was noted continuously, and excessive temperature-rise was prevented by watering. It was observed that heating proceeded more slowly in high heaps than in stacks of less height. This is contrary to the prevalent view that the danger of spontaneous combustion is less in the case of low heaps. It was also observed that after 5 months of storage there was practically no loss in calorific value or in the quantity of gas that would be produced.

Shaft-Sinking in Quicksand. E. S. Tillinghast (**Engng. & Min. J.*, 141 (5), *37–39; May 1940*).—The Author describes in detail the method employed at a mining property where several shafts had to be sunk through quicksand over a period of years. The most important feature of the method is a "shoe" formed of heavy steel plate, which is forced on ahead of actual digging. The height of the shoe depends upon the thickness of quicksand expected, the depth at which it is located, and the consequent water-pressure which results. This height may range from 2 feet for a thin layer of sand to 6 feet or more where 30–50 feet of water-saturated quicksand has to be traversed. The precautions to be observed in sinking are discussed, and the necessity for the exercise of care is emphasized.

The Atmospheric Electricity, an Overlooked Quantity in Mine Ventilation Air. L. Funder (*Glückauf*, 76, *237–243; 27 Apr. 1940*).—The Author has investigated the conditions in which atmospheric electricity is present in the ventilation air of a mine and has determined the ion contents of the air in coal, potash, and metalliferous mines. He discusses the causes of ionization and of de-ionization, and their possible relation to climatic conditions underground, to the results of geophysical investigations, to the development of firedamp and coal-dust explosions, and to the incidence of silicosis. He also examines the influence of newly-formed coal and potash dusts, as well as of the products of combustion of explosives, and discusses the ionization of the air produced in the course of various mining operations.

The Determination of Dust-Concentrations in Mine Atmospheres. J. H. Griffiths and T. D. Jones (**Trans. Instn. Min. Engrs.*, 99, *150–165; July 1940*).—In the 29th Report to the Committee on the Control of Atmospheric Conditions in Mines, the Authors discuss the problem of

collection of the dust actually present in the air, and describe a new methoc of sampling the dust, developed at University College, Cardiff. The apparatus comprised a wind-tunnel and a dust-cloud-producer consisting of a compressed-air injector, the suction-leg of which dipped into a tal cylindrical dust-reservoir which was raised slowly at a uniform rate by means of an automatically-adjusted counterbalance weight fitted over a pulley driven by a synchronous motor. The volume of compressed ai was measured by a gate-type meter, and from these figures the volume o the dust-cloud was calculated. The results are presented in Tables anc curves.

Endless-Rope Haulage : Large Installations working to the Dip E. H. Browne (**Trans. Instn. Min. Engrs.*, 99, *122–138 ; June 1940*).— The Author discusses the over-rope type of endless haulage adopted i cases where the roadways follow seams which have such variations i gradient as to preclude the under-rope system of haulage. He emphasize the desirability of proper planning and arrangement of the workings, a the concentration of haulage roadways and the limitation to a few feeder are essential requirements for economical working. He discusses in detai seven schemes for endless-rope haulages ranging from 70 horse-power t 250 horse-power, describes the haulage gear and the electrical equipment and considers also the track, the ropes and the method of their attachment and the running operations.

Slow Bankers. T. G. Dash (**Min. Elect. Engr.*, 20, *342–350 June 1940*).—The Author discusses the installation, maintenance, an testing of slow banking equipment as required by the Mines Department and describes briefly two types, the operation of which depends, respec tively, upon the action of a centrifugal governor or upon hydraulic actio All are equipped with accurate devices for measuring distance or trav and speed. The effects of temperature upon the operation of slow banke are considered, and the results of tests made with various types are plotte in curves. The Author emphasizes the necessity for adequate brake equip ment, and for the maintenance of its design performance.

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NOTE.—*Pages* [1] *to* [8] *can be omitted when the Journal is bound in volume form.*

NOTICES

No. 1, 1940—41

NOVEMBER, 1940

MEETINGS, SESSION 1940–41.

ORDINARY MEETINGS.

Arrangements have been made for the following subjects to be brought forward in the first part of the session, on the dates shown below :—

1940.
(1.30 p.m.)

Nov. 19.* The Dugald Clerk Lecture. "**Methods of Excavation Work at Home and Abroad,**" by William Barnes, M.I. Mech. E.

There will be a ballot for the election of new members.

Dec. 17.* "**The Mohammad Aly Barrages,**" by A. G. Vaughan-Lee, M. Inst. C.E.

There will be a ballot for the election of new members.

(*Light refreshments will be served at* 12.45 *p.m.*)

SPECIAL ANNOUNCEMENTS.

MILITARY SERVICE. AIR MINISTRY. MINISTRY OF LABOUR.

Details of the following appear on pp. [1]–[4] respectively of the October, 1940, Number of the Journal :—

Registration in the Army Officers' Emergency Reserve; special enlistment in the Royal Engineers; and entry into the ranks of the Royal Engineers under the National Service (Armed Forces) Act, 1939 (pp. [1]–[2]).

Commissions as technical (engineer) officers in the Royal Air Force Volunteer Reserve (pp. [2]–[3]).

* Brief abstracts appear on pp. [7], [8].

An entry in the Ministry of Labour's Schedule of Reserved Occupations as affecting "student engineering apprentices or learners" who wish to sit for Sections A and B of the Associate Membership Examination; together with information as to the procedure to be adopted by Corporate Members of The Institution who are required to register at Local Employment Exchanges when their age-groups are called up under the National Service (Armed Forces) Act, 1939 (pp. [3]–[4]).

MINISTRY OF LABOUR AND NATIONAL SERVICE.

The following letter has been received from the Ministry of Labour and National Service :—

Ministry of Labour and National Service,
International Labour Branch,
Hanway House,
Red Lion Square,
London, W.C.1.
9th October, 1940.

The Secretary,
The Institution of Civil Engineers,
Great George Street,
S.W.1.

DEAR SIR,

I am sending you herewith an extract from the speech made by Mr. Bevin, Minister of Labour and National Service, at a meeting of the Works Management Association held in London on 18th September.

As you will see from this extract, there are in this country a not inconsiderable number of engineering technicians and specialists of allied nationalities who are anxious to put their services at the disposal of this country in support of the common war effort.

I hope it will be possible for your Council to publish this extract in their Journal for the information of members who might be able to avail themselves of the services of any of these allies of ours.

Any further particulars concerning the qualifications of the men available can be obtained from the International Labour Branch of this Ministry at the above address. All the highly qualified specialists in the engineering and allied trades to whom the Minister referred in his speech are registered with the Central Register of Aliens kept in this Department.

Yours faithfully,
(signed) T. T. SCOTT.

EXTRACT from the Minister's Speech to the Works Management Association on the 18th September, 1940.

"... *International Labour Force.*

"Another matter on which I need your assistance is in connection with our Allies. You know that country after country has been mown down by Hitler and many of their workpeople, technicians, craftsmen and men of their mercantile marine are in this country, and the Government decided to establish an International Labour Force. You have already read of the results of General de Gaulle's activities, of the exploits of the Polish airmen, the Czech airmen, the Norwegian seamen and our Dutch and Belgian friends. These people are fighting and giving their lives in the defence of

the great cause—they are not refugees or aliens, they are our equals and we cannot leave unused their ability, skill and energy. Would Hitler have left them unused in his country had they remained there and not fought against him? He would have used every possible device to exploit their skill. In this Force we have already registered a large number of these international friends. They include chemical, electrical, aeronautical, mechanical, mining and other engineers, industrial chemists and industrial research workers. There is also a number of craftsmen and people with experience of particular trades, and we want to see their services utilized in such a way that they are in fact making their contribution towards the equipment of their own Forces as well as to the common pool. The Department which is being operated by General Appleyard and Mr. Scott would welcome assistance from you in the absorption of these technical and skilled workers.

"It should be clear that this Department only deals with people whom the Ministry of Home Security have certified as being all right. So far as the Government is concerned, our policy has been from the point of view of social services and other State action, to treat them as equals with ourselves. . . ."

GENERAL ANNOUNCEMENTS.

THE JOURNAL.

The next Number will be published on the 15th December.

SERVICE IN THE FORCES.

For office purposes, a record is being kept of members' service with H.M. Forces, and members who have not already done so are asked to inform the Secretary of such service, i.e. unit, rank, promotions, decorations, etc. Further, practical use is made of such information when inquiries from the Services are received by The Institution.

EVACUATION OF CHILDREN TO CANADA.

The Government having announced that they cannot take the responsibility of sending children overseas under the Government evacuation scheme during the winter season, it is regretted that it is not possible for the present to take advantage of the generous offer of Canadian engineers to provide homes for the children of British engineers. It is hoped that conditions may render it possible to resume the scheme next year, and, in the meantime, members who have registered, or who are able to register, their children with the Children's Overseas Reception Board for evacuation to Canada, or who contemplate sending their children privately, and who have no relatives or friends in that country to whom to send them, are asked to communicate full particulars to the Secretary Inst. C.E.

HONOURS.

The Council have much pleasure in congratulating the following Members on the Distinctions conferred upon them :—

BARON.

Reith of Stonehaven, *The Rt. Hon. Baron*, P.C., G.C.V.O , G.B.E., M.P., D.C.L., LL.D., M.Sc. — *Member.*

O.B.E. (Military Division).

McCreary, Jun., Lt.-Col. Robert, M.C., B.Sc., R.E. — ,,
"For gallantry."

ABSTRACTS.

The publication of "Engineering Abstracts" in sectional form was suspended in September, 1939. A selection of brief Abstracts of important Papers and articles appearing in the home and foreign technical literature dealing with Engineering Construction, Mechanical Engineering and Mining Engineering is now included in the Journal.

Abstracts of the technical press on Shipbuilding and Marine Engineering (formerly Section 3 of "Engineering Abstracts") are compiled by the Institute of Marine Engineers and members are able to obtain these at half the usual subscription rates.

Members are also able to obtain copies of "Building Science Abstracts" compiled by the Building Research Station, Watford, and of the "Summary of Current Literature" issued by the Water Pollution Research Board, at the special rates detailed below, provided that the order is placed through the Secretary of The Institution.

The subscription rates for 1941 are as follows :—

Shipbuilding and Marine Engineering Abstracts	12*s*. 6*d*. (post free).
Building Science Abstracts	14*s*. 6*d*. (post free).
Water Pollution Research : Summary of Current Literature	18*s*. 6*d*. (post free).

All subscriptions run from January.

ROAD ABSTRACTS.

The question of the continuance of the publication of Road Abstracts is under consideration. A further announcement will be made in the December issue of the Journal.

TRANSFERS.

On the 24th September, 1940, the Council transferred two Associate Members to the class of Members.

DEATHS AND RESIGNATIONS.

The Council have received, with regret, intimation of the following deaths and resignations :—

Deaths.

Hadfield, Sir Robert Abbott, Bart., D.Sc., D.Met., F.R.S. (*Former Vice-President.*) (Elected *A.M.* 1887.) (Transferred *M.* 1896.) (Elected *Hon.M.* 1937.)	*Hon. Member.*
Barratt, Charles Henry. (E. 1907.)	*Member.*
Blaker, William Herbert. (E. 1904. T. 1929.)	,,
Buxton, Murray Barclay, M.C., M.A. (E. 1923. T. 1940.)	,,
Martin, Arthur John. (E. 1898. T. 1906.)	,,
Merz, Charles Hesterman. (E. 1905.)	,,
Morgan, Reginald Travers, M.Eng. (E. 1919. T. 1931.)	,,
Morley, John George. (E. 1889. T. 1911.)	,,
Reid, Andrew Thomson. (E. 1904.)	,,
Brierley, John Henry. (E. 1894.)	*Associate Member.*
Martin-Leake, Stephen. (E. 1888.)	,, ,,
Shields, Alfred Henry Thomas. (E. 1930.)	,, ,,
Whinfield, John Henry Richard. (E. 1885.)	,, ,,
Angus, Alexander Robert. (A. 1940.)	*Student.*
Raikes, Peter Patrick. (A. 1934.)	,,

Resignations.

McKenzie, Charles John, C.B.E. (E. 1916.)	*Associate Member.*
Goodwin, Frederick Dale, B.A. (A. 1936.)	*Student.*
Green, Wilfrid Garner. (A. 1937.)	,,
MacDiarmid, Colin Pitblado. (A. 1937.)	,,
Wood, James. (A. 1939.)	,,

RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, etc., are not included.]

Aircraft. Brimm, D. J., and Boggess, H. E. "Aircraft Maintenance." 1940. Pitman. 10*s*. 6*d*.

Atmospheric Pollution. Department of Scientific and Industrial Research. "The Investigation of Atmospheric Pollution. 25th Report, 1938–9." 1940. H.M.S.O. 2*s*. 6*d*.

Balloons. Sumner, P. H. "Design and Stability of Streamline Kite Balloons." 1920. Lockwood. 10*s*. 6*d*.

Chemistry. Gyngell, E. S. "Applied Chemistry for Engineers." 1940. Arnold. 15*s*.

Concrete. Chain Belt Company. "Pumpcrete Practice. A Manual of Concrete Placement by Pumpcrete." 1940. The Company, Milwaukee. No price.

Cordage. Stopford, P. J. "Cordage and Cables." 2nd ed. 1940. Brown, Son, and Ferguson. 5*s*.

—— Woodhouse, T., and Kilgour, P. "Cordage and Cordage Hemp and Fibres." 1919. Pitman. 3*s*.

Cyclotron. Mann, W. B. "The Cyclotron." 1940. Methuen. 3*s*.

DIESEL ENGINES. RICHARDS, E. L. "Diesel Engines and Diesel Electric Power." 1940. Pitman. 10*s*. 6*d*.

ELECTRICAL TROUBLES. WATTON, E. B. "How to prevent Electrical Troubles." 1940. Marshall. 2*s*. 6*d*.

ENGINEERING. LOW, A. M. "The Way It Works." 1940. Davies. 8*s*. 6*d*.

GASES. JEANS, *Sir* J. "Introduction to the Kinetic Theory of Gases." 1940 Cambridge University Press. 15*s*.

GAUGES AND GAUGING. WAY, R. B. "Gauges and Gauging." 1940. Marshall 2*s*.

GEOLOGY. RUNNER, D. G. "Geology for Civil Engineers, as related to Highway Engineering." 1939. Gillette Publ. Co. 5 dollars.

HEATING. DILWORTH, J. L. II. "Radiator Capacities in Terms of Heating Loads."

—— JOHNSTON, R. M. I. "A Graphical Solution of Heating Loads." Bulletin No 45, Engineering Experiment Station, Virginia Polytechnic Institute, Blacksburg Va. 1940. No price.

HYDRAULICS. ADDISON, H. "Hydraulic Measurements." 1940. Chapman and Hall. 21*s*.

—— POWELL, R. W. "Mechanics of Liquids." 1940. Macmillan Co. 15*s*.

IRRIGATION. INDIA. CENTRAL BOARD OF IRRIGATION. "Annual Report (Technical) 1938–9." 1940. Govt. of India Press, New Delhi.

METALS. BOLAS, T. "Soldering, Brazing, and Joining of Metals." Revised ed 1940. Marshall. 1*s*. 6*d*.

—— JACKSON, C. F., and HEDGES, J. H. "Metal Mining Practice." (U.S. Bureau of Mines Bulletin No. 419.) 1939. Supt. of Documents, Washington. 3*s*. 6*d*.

MILITARY SCIENCE. PORTWAY, Lt.-Col. D. "Military Science To-day." 1940 Oxford University Press. 8*s*. 6*d*.

MINERAL RESOURCES. DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH GEOLOGICAL SURVEY OF GREAT BRITAIN. "Synopsis of the Mineral Resources of Scotland." 1940. H.M.S.O., Edinburgh. 1*s*.

—— UNION OF SOUTH AFRICA DEPT. OF MINES. "The Mineral Resources of the Union of South Africa." 1940. Govt. Printer, Pretoria. 5*s*.

PLASTICS. SIMONDS, H. R. "Industrial Plastics." 1940. Pitman. 22*s*. 6*d*.

PRODUCTION. MITCHELL, W. N. "Organization and Management of Production." 1930. McGraw-Hill. 26*s*.

ROADS AND STREETS. MICHIGAN UNIVERSITY. "Proceedings of the 25th Anniversary Highway Conference." 1940. The University, Ann Arbor. No price.

—— U.S. HIGHWAY RESEARCH BOARD. "Proceedings, 19th Annual Meeting Vol. 19, 1939." National Research Council, Washington. 15*s*.

RUBBER. WOOD, L. A. "Synthetic Rubbers: a review of their composition properties, and uses." (Circ. C. 427, U.S. Bureau of Standards.) 1940. Govt Printing Office, Washington. 10 cents.

TELEVISION. FINK, D. S. "Principles of Television Engineering." 1940. McGraw Hill. 33*s*.

VAPOUR CHARTS. ELLENWOOD, F. O., and MACKEY, C. O. "Vapor Charts and Special Tables for Turbine Calculations." 1939. Chapman and Hall. 15*s*.

LOCAL ASSOCIATIONS.

YORKSHIRE ASSOCIATION.

v. 23.—Meeting. Repetition of the Dugald Clerk Lecture on "Methods of Excavation Work at Home and Abroad," by William Barnes, M.I.Mech.E. (Leeds, 2.30 p.m.).

THE DUGALD CLERK LECTURE, 1940–41.

Date of Reading 19/11/40.

"Methods of Excavation Works at Home and Abroad."

By William Barnes, M.I. Mech. E.

Synopsis.

ɪᴇ Lecture will present brief descriptions of the many types of excavators w available, and of their application on canals, including the Suez ɑal, the Manchester ship canal, the Panama canal, and the latest large ip canal, the Albert canal in Belgium; on drainage and irrigation ıemes, including the Sukkur scheme in India—the largest irrigation ıeme ever carried out; in opencast mining and quarrying, at the iron mines of Kiruna (Sweden), Iron Knob (Australia), Lake Superior .S.A.), and at iron ore mines and ironstone quarries in England; and opencast coal mines in the United States and the Fushun coal mines, ınchukuo.

Descriptions will also be given of the latest developments in tractor uipment, including scrapers, graders, bulldozers, etc., and of typical plications at home and abroad.

ABSTRACT OF A PAPER FOR DISCUSSION.

The following Paper will be brought forward for discussion on the te indicated in the margin of the abstract, and will be published, with ports of the oral and written discussions upon it, in the Journal. mbers desiring to take part in the consideration of this Paper should ply forthwith for advance copies, which will be forwarded as soon as ey are ready. Applications for proofs should be made on postcards, ting the number of the Paper.

A period of about 3 months from the date of publication of the Paper the Journal is generally allowed for written communications, which uld be :—

(*a*) As concise as possible and entirely relevant to the subject-matter of the Paper;

(*b*) Written legibly or typed with the lines openly spaced.

Date of Discussion 17/12/40.

Paper No. 5251.

"The Mohammad Aly Barrages, Egypt."

By Alec George Vaughan-Lee, M. Inst. C.E.

The Paper reviews the history of the Delta barrages, across the Damie and Rosetta branches at the southern apex of the Nile delta, and descri the design and construction of the new Mohammad Aly barrages wh have been constructed downstream from them.

Detailed descriptions are given of the principal items of the wo including reconditioning of the Tewfiki regulator and lock, the r Damietta barrage and navigation lock, a hydraulic research laborato a new intake and regulator for the Nagayel canal, the new Rosetta barr and lock, a new diversion for the Behera canal, with a regulator a navigation lock, and about 4 kilometres of roads. The cost of the wo was about £E2,405,000.